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DISCUSSIONS

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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

FORMATION OF FLOC BY FERRIC COAGULANTS

BY EDWARD BARTOW,[1] M. AM. SOC. C. E., A. P. BLACK,[2] ESQ., AND WALTER E. SANSBURY,[3] ESQ.

SYNOPSIS

Ferrous sulfate (oxidized by atmospheric air in the presence of alkali), ferric chloride, chlorinated copperas, and ferric sulfate are used as coagulants in water purification. Natural waters contain varying quantities of sulfate ion, chloride ion, sodium ion, calcium ion, etc. An experimental determination of the action of these ions on the formation of floc with ferric salts at different pH-values shows the following results: (1) On the acid side the sulfate ion has a much greater effect on coagulation than the chloride ion; (2) using from 25 to 250 ppm of sulfate ion, there is little change in the effect produced; (3) between a pH-value of 6.5 and a pH-value of 8.5, there is a zone in which ferric floc forms slowly or not at all; and (4) in and beyond this zone, sodium and calcium ions are most effective in coagulation.

The assumption of a change in the sign of the colloidal ferric floc from positive, where it is more affected by sulfate or chloride ions, to negative will explain the zone of no floc formation and the more effective action of the sodium and calcium ions beyond this zone.

The quantity of residual iron in solution is roughly proportional to the time required for the floc to form.

FERRIC COAGULANTS

It has long been recognized that ferric compounds may be utilized in the formation of floc for the removal of color or turbidity from natural waters, but it is only in recent years that they have become available in adequate quantity and at a price sufficiently low to compete with alum.

With the possibility of increased use of these ferric coagulants, the writers have undertaken a study similar to that made by Messrs. Bartow and

NOTE.—Discussion on this paper will be closed in March, 1934, *Proceedings*.

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Ben. H. Peterson[4] and by Messrs. Black, Rice, and Bartow[5] on aluminium sulfate. It is known that aluminium salts and iron salts differ in their action when used as coagulants in water treatment, and the purpose of this paper is to try to determine and explain these differences.

Ferrous Sulfate as a Coagulant.—Ferrous sulfate with lime was first used by Mr. W. B. Bull, at Quincy, Ill. According to Mr. Dow R. Gwinn[6] the experiments were begun in July, 1898, the ferrous sulfate being made by burning sulfur and passing the gas into water and over scrap iron. Chlorinated copperas made by adding chlorine to a solution of ferrous sulfate was used by Messrs. F. W. Mohlman and J. R. Palmer,[7] at Chicago, Ill., in the coagulation of sewage sludge. It has since been adapted to water purification.

Since the coagulation takes place after oxidation to ferric salt by the oxygen of the air, ferrous sulfate is really a ferric coagulant. It is unsuitable for the treatment of certain types of waters. Soft, colored waters, or soft, turbid waters, are best coagulated on the acid side of the neutral point—at pH-values of less than 7. (pH is equal to the log of the reciprocal of the hydrogen-ion concentration.) The color appears to become fixed or set by the addition of alkali. The use of ferrous sulfate is limited, therefore, to those waters in which alkalinity will not interfere with color removal. It has been shown by Mr. L. B. Miller[8] and, later, by Messrs. Jacob Cornog and Albert Hershberger[9] that when alkali is added to a dilute solution of ferrous sulfate, a small fraction of one equivalent increases the pH to about 7.5. It remains practically constant at this figure until nine-tenths of an equivalent of alkali has been added; then it rises again rapidly to a pH of about 11.0. Further addition of alkali up to the theoretical two equivalents produces little change in the hydrogen-ion concentration. According to Mr. Miller (see Fig. 1), the first zone of fairly constant hydrogen-ion concentration is due to the formation of a basic salt. It is obvious, therefore, that ferrous salts are precipitated by alkali only at comparatively high pH-values.

A more important factor is the atmospheric oxidation of the ferrous coagulant to the ferric state. Messrs. J. A. N. Friend and E. G. K. Pritchett[10] and Messrs. Cornog and Hershberger[9] have shown that the rate of oxidation of ferrous salts by atmospheric oxygen depends upon both the concentration of the ferrous salt and the hydrogen-ion concentration. For the exceedingly low concentrations met with in water-works practice, the rate of oxidation increases as the hydrogen-ion concentration decreases; that is, the rate increases with increased pH-values.

Mr. C. V. Smythe[11] has shown that certain anions greatly influence the rate of atmospheric oxidation of ferrous compounds. The theory is advanced

[4] *Industrial and Engineering Chemistry*, Vol. 20 (1928), p. 51.

[5] *Loc cit.*, Vol. 25 (1933), p. 811.

[6] *Engineering News*, Vol. 43 (1900), p. 351; *Engineering Record*, Vol. 39 (1899), p. 595.

[7] *Engineering News-Record*, Vol. 100 (1928), p. 147.

[8] Public Health Repts., U. S. Public Health Service, Vol. 40 (1925), p. 1413.

[9] Thesis for Master's Degree, State Univ. of Iowa, Iowa City, Iowa, 1928.

[10] *Journal*, Chemical Soc., 1928, p. 3227.

[11] *Journal of Biological Chemistry*, Vol. 95 (1931), p. 251.

that the ferrous ion forms unionized complexes with the anions in question and that it is easier to remove an electron from such a neutral complex than from a doubly charged ferrous ion. In terms of this theory the oxidation of ferrous salts in alkaline solution would involve the formation of insoluble

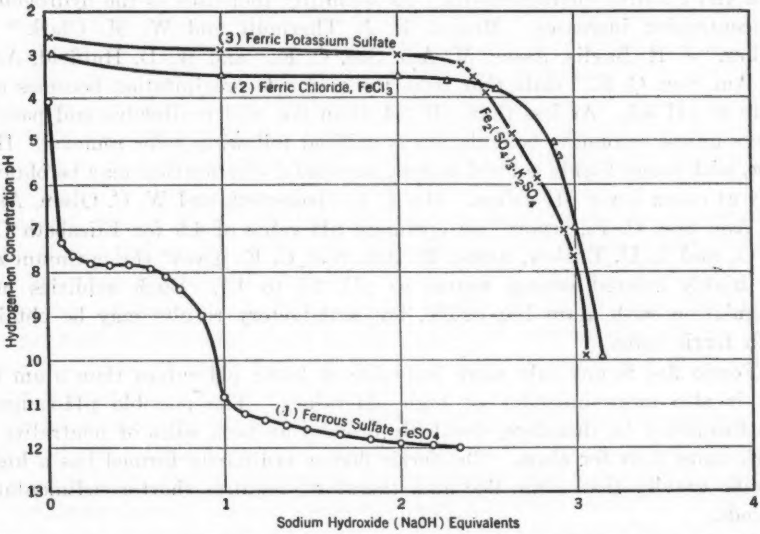

FIG. 1.—COMPARISON OF CHANGE IN HYDROGEN-ION CONCENTRATION (pH), DURING PRECIPITATION OF FERROUS HYDROXIDE AND FERRIC HYDROXIDE BY SODIUM HYDROXIDE.

neutral molecules of ferrous hydroxide which are far more readily oxidized than the ferrous ion. These considerations also apply in the removal of iron from natural waters by aeration.

Ferric Salts as Coagulants.—Ferric chloride made by treating scrap iron with gaseous chlorine obtained directly from a chlorine cell was used experimentally in 1910 in Chicago and in Toledo, Ohio, by Mr. Bull.[12] Ferric chloride in concentrated solution, shipped in rubber-lined tank cars, has been available since about 1930. Ferric sulfate in dry form was placed on the market in 1931.

In coagulation or formation of floc, the action of ferric salts is very different from that of ferrous sulfate. According to Mr. Miller[8] a very dilute solution (0.01 M) of ferric sulfate, when treated with an alkali, has a pH-value of 3 when two equivalents of alkali have been added; of 4 when two and one-half equivalents have been added; and of 7 when the three equivalents theoretically required have been added. (The symbol, M. denotes mols, or molecules per liter.) With ferric chloride the pH-values are slightly higher. (See Fig. 1.) This means that a ferric floc may be formed in solutions that are much more acid than would normally be encountered in water-works practice.

[12] *Proceedings*, Illinois Water Supply Assoc., Vol. 3 (1911), p. 66; and *Municipal Engineering*, Vol. 42 (1911), p. 159.

Since soft, colored waters and soft, turbid waters may usually be coagulated more satisfactorily on the acid side, alum or a combination of alum and sulfuric acid has been used satisfactorily as a coagulant with such waters. However, since aluminium hydroxide is amphoteric (that is, exhibits both acid and alkaline characteristics,) its solubility increases as the hydrogen-ion concentration increases. Messrs. E. J. Theriault and W. M. Clark,[13] Mr. Miller,[14] J. R. Baylis, Assoc. M. Am. Soc. C. E.,[15] and W. D. Hatfield, Assoc. M. Am. Soc. C. E.,[16] state that from the acid side precipitation becomes complete at pH 5.4. At less than pH 5.4 alum floc will re-dissolve and pass the filters unless secondary coagulation is utilized following color removal. However, with many highly colored waters, successful clarification may be obtained only at much lower pH-values. Mr. L. L. Hedgepeth and W. C. Olsen, Assoc. M. Am. Soc. C. E., report[17] an optimum pH-value of 4.9 for Elizabeth City, N. C., and L. H. Enslow, Assoc. M. Am. Soc. C. E., gives[18] the optimum zone for highly colored swamp waters as pH 3.8 to 4.7. Such acidities make coagulation with alum impossible, but satisfactory results may be obtained with ferric salts.

Ferric floc is not only more insoluble at lower pH-values than alum floc, but is also more insoluble at high pH-values. The possible pH-range of floc formation is, therefore, decidedly broader on both sides of neutrality for ferric salts than for alum. The ferric floc as ordinarily formed has a higher specific gravity than alum floc and, therefore, requires shorter sedimentation periods.

Explanations differ as to the exact mechanism of the process by which turbidity and certain types of color are removed. While it has long been thought that the highly adsorptive gelatinous floc plays the leading rôle, Mr. Miller[19] in his fundamental work indicates that color removal is most often accomplished by neutralization of the negatively charged color colloid by the positively charged trivalent aluminium or ferric ion and that as a rule "alum floc" and "iron floc' play relatively unimportant parts.

Effect of Various Anions on Floc Formation

Waters apparently similar in character may respond in very different ways to the same method of treatment. This is particularly true of the hydrogen-ion concentration, at which optimum floc formation is secured with a given dose of coagulant. The point of optimum coagulation may be as low as pH 3.8, in the case of highly colored swamp waters, or it may be greater than 8.0 with other types of waters. It is often true that the pH-range within which good flocs may be formed rapidly is relatively narrow and in some cases the pH-values must be adjusted closely.

[13] Public Health Repts., U. S. Public Health Service, Vol. 38 (1923), p. 181.
[14] *Loc. cit.*, p. 1995; and Vol. 40 (1925), p. 351.
[15] *Journal*, Am. Water Works Assoc., Vol. 10 (1923), p. 365.
[16] *Loc. cit.*, Vol. 11 (1924), p. 554.
[17] *Loc. cit.*, Vol. 20 (1928), p. 467.
[18] *Proceedings*, First Virginia Conference on Water Purification and Sewage Treatment, 1929, and *Municipal News and Water Works*, Vol. 76 (1929), p. 227.
[19] Public Health Repts., U. S. Public Health Service, Vol. 40 (1925), p. 1472.

The investigations of Messrs. Bartow and Peterson[4] and of Messrs. Black, Rice, and Bartow[5] have explained in part at least the "individuality" possessed by most natural waters. These investigations have shown that the anions present in the water treated, particularly the sulfate and chloride ions, influence to a marked degree the pH-zone of optimum floc formation when alum is used as a coagulant. The bivalent sulfate ion extends the pH-zone of optimum floc formation far to the acid side and very slightly to the alkaline side, the effect increasing, as would be expected, with increasing quantities of sulfate ion. The monovalent chloride ion has much less influence than the bivalent sulfate ion. It does broaden the pH-zone somewhat, and it is to be noted that its effect is somewhat greater on the alkaline side than that of the sulfate ion.

Messrs. Black, Rice, and Bartow,[5] at the experimental water treatment plant of the University of Florida, have shown that results obtained in jar tests are likewise obtained when large quantities of water are treated.

In view of the marked effects exerted by the sulfate and chloride ions on the pH-zone of optimum alum floc formation, it seemed desirable to determine the effect of these ions on the formation of ferric floc with various ferric coagulants.

Three ferric coagulants are available to-day—chlorinated copperas, ferric chloride, and soluble ferric sulfate—although chlorinated copperas is not sold as a ferric salt. It is a mixture of ferric chloride and ferric sulfate obtained by oxidizing ferrous sulfate (copperas) with chlorine; hence its name. The reaction is as follows: $6FeSO_4 + 3Cl_2 \rightarrow 2Fe_2(SO_4)_3 + 2FeCl_3$.

In practice, 1 lb of chlorine is required for 8 lb of copperas. As stated previously, chlorinated copperas instead of ferric chloride was first used by Messrs. Mohlman and Palmer[7] for dewatering sewage sludge. It proved greatly superior to alum. It was first used in water purification by Messrs. Hedgepeth and Olsen[17] in 1928 for the treatment of the highly colored water supply of Elizabeth City, N. C. The so-called "color floc" produced at low pH-values by the action of the trivalent ferric ion on the negatively charged color colloid was stated by them to have adsorptive properties far superior to those of a true ferric hydroxide floc. Following its highly successful use at Elizabeth City, trials were made elsewhere, and several papers describing the results obtained are available in the literature. A. C. Decker, M. Am. Soc. C. E., and Mr. H. G. Menke[20] used the process at Birmingham, Ala. Mr. E. S. Hopkins,[21], at Baltimore, Md., studied the floc produced by this material.

Messrs. E. C. Craig and E. H. Bean,[22] at Providence, R. I., have used the three ferric coagulants interchangeably and conclude that there is less than 10% variation in the results obtained with any of them when they are compared on the basis of equal iron content.

[20] *American Journal of Public Health,* Vol. 20 (1930), p. 357.; and *Municipal News and Water Works,* Vol. 76 (1929), p. 246.

[21] *Industrial and Engineering Chemistry,* Vol. 21 (1929), p. 581, and Vol. 22 (1930), p. 79.

[22] *Water-Works and Sewerage,* Vol. 79 (1932), p. 301.

Experimental

Coagulation experiments have been made in the stirring apparatus used by Messrs. Black, Rice, and Bartow[5] in the study of the effect of various anions on the formation of aluminium floc.

For these experiments six battery jars (3.5 liter capacity) were used; 2 liters of water were used for each test. Glass stirrers were used to mix the water in the battery jars. They were composed of glass blades, each 3 by 15 cm., attached to vertical glass rods, 1 by 25 cm., which, in turn, were attached to vertical steel drive shafts by heavy walled rubber tubing, so that each might be disconnected and the jars removed during a run. The vertical shafts were supported on an iron framework and on the upper end had pulleys, all driven by one continuous belt from a small electric motor. Identical conditions of illumination and the formation of Tyndall cones were produced by rays from 100-watt electric light bulbs, controlled by a single switch, the light from which passed through $\frac{3}{8}$-in. holes in a long wooden box placed behind the jars. The Tyndall cones produced were so sharp that it was usually possible to duplicate the shorter floc times to 15 sec in successive experiments. To exclude heating effect from the bulbs, the side of the box nearest the jars was lined with asbestos board which, together with a 1-in. air gap between the box and the jars, caused a maximum rise of only 1°C during the longest runs. The apparatus with the exception of the motor and gear reducer was built in the machine shops of the University of Florida at little cost. G. E. Willcomb, Assoc. M. Am. Soc. C. E., describes[23] the construction of laboratory stirring machines of this general type.

Method of Procedure.—The determined quantities of alkaline solution and salt solution to be used, were placed in a 2-liter volumetric flask, made up to a volume of 1 975 milliliters, with distilled water, thoroughly mixed and placed in the jars. Stirring was begun and 25 milliliters of coagulant solution was added to each jar. The time of addition was noted with a stopwatch. The time at which well defined floc formed in each jar was carefully noted, and the total elapsed time for each jar was corrected for the time required to add the coagulent.

The equivalent of 1.6 grains of coagulant, calculated as $Fe_2(SO_4)_3 \cdot 9H_2O$, was used for each test. This is the approximate molar equivalent of the 2 grains of alum used in previous work. The quantities of alkali in the various tests required to adjust the solutions to the desired pH-values, were determined by experience.

Standard Solutions.—The purest analytical grade of chemicals was used for the preparation of all solutions, which were made of such strength that convenient small volumes, when diluted to a final volume of 2 liters, gave known concentrations of the desired constituent. In the case of the coagulants, 25 milliliters of working solution represented a dose of 1.6 grains per gal., calculated as ferric sulfate. The solutions of sodium chloride, sodium sulfate, sodium bicarbonate, and calcium chloride were of such strength that each milliliter contained 10 mg of the respective anion. The chlorinated

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[23] *Journal*, Am. Water Works Assoc., Vol. 24 (1932), p. 1416.

copperas was prepared by passing chlorine gas through a ferrous sulfate solution until oxidation was complete, as shown by testing with potassium ferricyanide. Air was then bubbled through the solution until starch iodide paper showed the absence of excess of free chlorine. Concentrated stock solutions of ferric sulfate and chlorinated copperas were prepared and the exact strengths determined by titration with standard potassium dichromate solution. Since the dilute working solutions were unstable, they were prepared from the stock solutions in small quantities as needed.

As soon as the floc had formed in a jar, this jar was removed from the machine without interrupting the stirring of the other jars. A portion of the water from the jar was placed in a clean Erlenmeyer flask, tightly stoppered, and allowed to settle until it was clear and bright. The pH-values of all solutions were determined by the isohydric colorimetric method fully described elsewhere.[5] Samples of solution (10 milliliters) were carefully pipetted into color comparison tubes, filled with air from which carbon dioxide had been removed, each containing 0.5 milliliter of an indicator solution of chosen pH-value. Thorough mixing was secured, without shaking, by allowing the water to flow from the pipette tip into the indicator solution in the tubes. The tubes were stoppered to exclude carbon dioxide and were then compared with buffer-color standards in a comparator. Ordinarily, the pH-value of each sample was determined with indicator solutions of three different pH-values. From the data a value for the hydrogen-ion concentration could be obtained which was reliable to 0.05 pH-unit.

Residual iron determinations were made by the thiocyanate method on samples of the water filtered through two thicknesses of ashless filter paper.

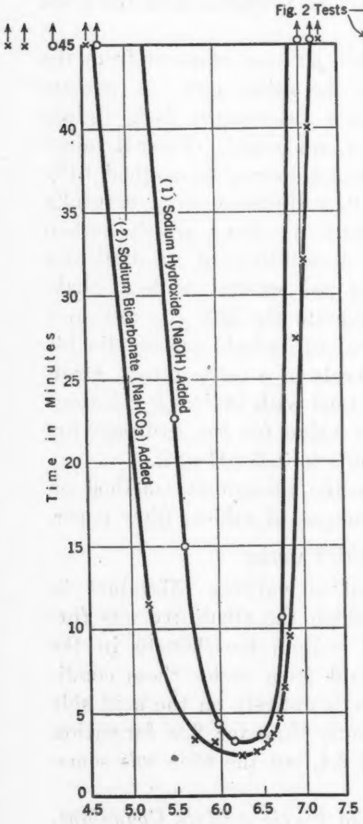
Floc Formation at Varying pH-Values

The time of floc formation was determined at varying pH-values in solutions of coagulant and distilled water, in which the alkalinity was furnished by sodium hydroxide in one case and sodium bicarbonate in the other. The pH-zone in which a ferric floc would form under these conditions was relatively narrow, pH 5.0 to 7.0. This is entirely on the acid side of the neutral point. (See Fig. 2.) The minimum time for floc formation occurred at about the same place, pH 6.1 to pH 6.4, but the zone was somewhat wider in the case of sodium bicarbonate.

Effect of Sulfate Ion on Floc Formation Using Ferric Sulfate Coagulant.—Sulfate ion as pure sodium sulfate, $Na_2SO_4 \cdot 10H_2O$, was added to the water in dosages of 25, 50, 125, and 250 ppm, and its effect on the pH-zone of floc formation determined. It was interesting to note: (1) That the zone of rapid floc formation was greatly widened on the acid side (Fig. 3, Curve 1). Floc formed in 4½ min at a pH-value of 3.9; (2) that the effect of a dose of 25 ppm of sulfate ion (see Curve 2, Fig. 3) was almost as great as that of a dose ten times as large (Curve 5), which was not found to be the case with alum[4]; (3) that the time of floc formation increased markedly in the pH-range, 7.0 to 8.5, after which it again decreased as the pH increased to the maximum used in this work, 9.6; and (4) that the solution containing 25 ppm of sulfate ion, which flocculated rapidly at all pH-values between 4.0

and 7.0, failed to flocculate in 45 min between pH-values of 7.0 and 8.0. This interesting and unexpected behavior of these solutions in this pH-range will be discussed later. Points marked with an arrow signify that no floc formed within 45 min at the designated pH-value.)

Effect of Chloride Ion on Floc Formation Using Ferric Sulfate Coagulant.—Chloride ion furnished by sodium chloride was added to the water in



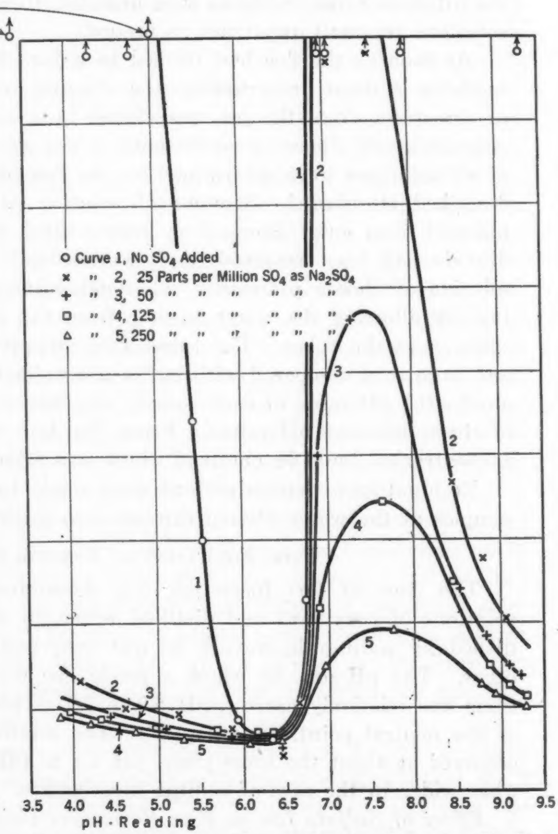


FIG. 2.—RATIO OF FLOC FORMATION BY FERRIC SULFATE WITH HYDROGEN-ION CONCENTRATION ADJUSTED BY: (1) SODIUM HYDROXIDE (NaOH) AND (2) SODIUM BICARBONATE ($NaHCO_3$).

FIG. 3.—EFFECT OF SO_4 AS SODIUM SULFATE (Na_2SO_4) ON THE ZONE OF FLOC FORMATION BY FERRIC SULFATE [$Fe_2(SO_4)_3$] AT VARYING pH-VALUES.

dosages of 25, 50, 125, and 250 ppm, and its effect on the pH-zone of floc formation determined in the same manner as for the sulfate ion. (See Fig. 4.)

On the acid side the chloride ion was not nearly as effective as the sulfate ion in influencing the formation of floc. While the zone of floc formation was greatly broadened on the acid side, the time required for the floc to form was considerably longer in all cases. This behavior is quite simply explained when it is remembered that the floc forms first as a positive

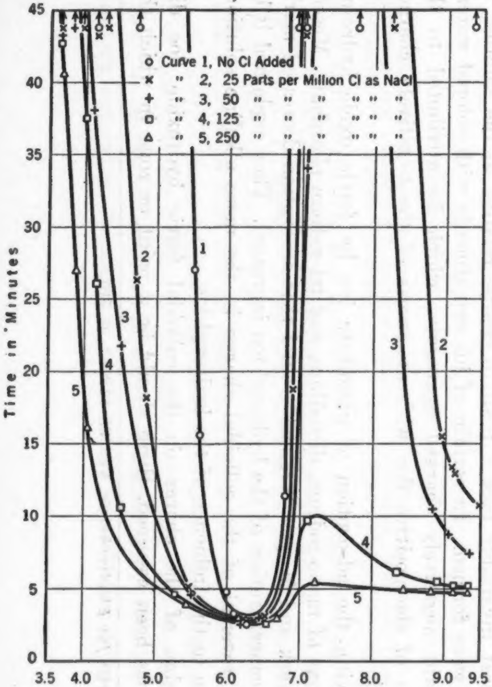


FIG. 4.—EFFECT OF CHLORIDE-ION AS NaCl ON THE ZONE OF FLOC FORMATION WITH FERRIC SULFATE.

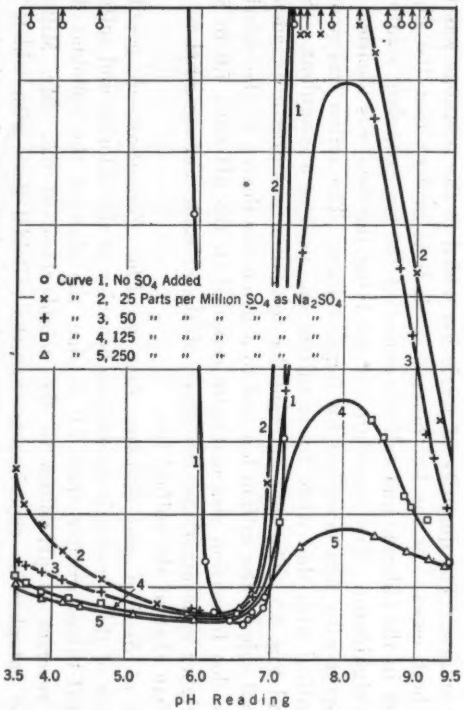


FIG. 5.—EFFECT OF SULFATE-ION AS Na_2SO_4 ON ZONE OF FLOC FORMATION WITH CHLORINATED COPPERAS.

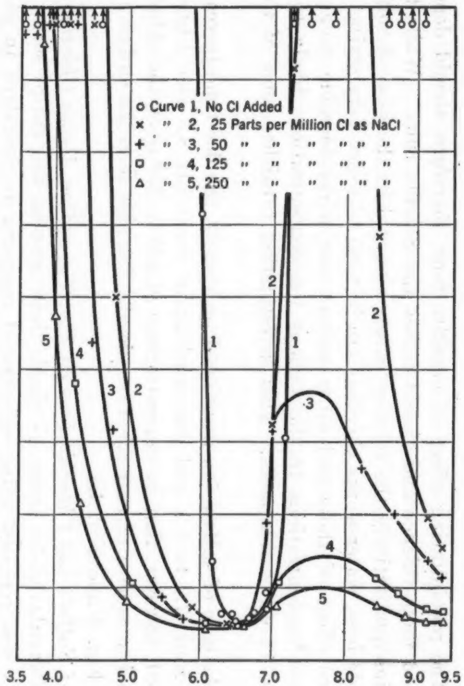


FIG. 6.—EFFECT OF CHLORIDE-ION AS NaCl ON ZONE OF FLOC FORMATION WITH CHLORINATED COPPERAS.

colloid. The precipitating effect of negative ions, therefore, should depend largely upon their valence and should follow the Schultz rule.[14] While this rule explains qualitatively the relative effects observed in the case of sulfate and chloride ions, water-works chemists are in need of exact data giving the quantitative effect of these and other anions over a considerable range of hydrogen-ion concentrations in the various ion concentrations likely to be encountered in water treatment. This investigation furnishes some of these data.

As in the case of sulfate ion mentioned previously, the floc time increased in the pH-zone, 7.0 to 8.5, and decreased again above pH 8.5 (see Fig. 3). It was noted, however, that in concentrations of 125 to 250 ppm, the chloride ion appeared to exert a greater effect on the alkaline side than the sulfate ion. (Compare Fig. 3 and Fig. 4.)

Effect of Sulfate and Chloride Ions on Floc Formation, Using Chlorinated Copperas Coagulant.—In order to check these results, the entire series of experiments was repeated using chlorinated copperas as coagulant in place of ferric sulfate. The quantity of ferric ion added to the solutions was the same in both cases. In the low dosages used, it is evident that the slight differences in the chloride and sulfate content of the two coagulant solutions were not significant. The procedure for obtaining the data was identical in every respect with that described for ferric sulfate. The action was practically identical with that when ferric sulfate was used as a coagulant. (See Fig. 5 and Fig. 6.) The zone of rapid floc formation was greatly broadened on the acid side by the sulfate ion and to a much less degree by the chloride ion. The floc formation was markedly retarded in the pH-zone, 7.0 to 8.5. At the higher concentrations, the effect of the chloride ion appeared to be greater than that of the sulfate ion.

Effect of Sodium and Calcium Ions on Floc Formation.—A possible explanation of this seemingly anomalous behavior of the chloride and sulfate ions at pH-values greater than 7.0 is that the sign of the colloidal floc particles reverses at pH-values in the neighborhood of 6.5. Mr. Miller[19] showed by cataphoresis experiments that the colloidal flocs with which he worked almost invariably bore a positive charge relative to the solution in which they were formed. In certain of his experiments with colored waters, he did obtain negatively charged aggregates which he attributed to the combination of the positive floc with an excess of the negatively charged color colloid.

In studying the adsorption of phosphate ion by ferric oxide hydrosols from solutions of mono-sodium, di-sodium, and tri-sodium phosphate, Messrs. W. Stollenwerk and M. von Wrabgel[24] found that adsorption decreased markedly as the concentration of the hydroxyl-ion increased. They explained it by assuming a reversal of the colloidal charge in the more alkaline solutions probably due to the influence of the hydroxyl-ion.

If the sign of the charge on the colloidal ferric hydroxide floc did reverse as has been assumed, there should be a point or zone in which the

[24] *Zeitschrift für Electrochemie*, Vol. 33 (1927), p. 501.

floc bears no charge. This should be the point of minimum stability at which precipitation should take place most rapidly. On the alkaline side of this point or zone, the cations present in solution, and not the anions, should act as precipitating agents. Sodium chloride contains approximately 40% of sodium, whereas anhydrous sodium sulfate contains only 32.4%, and the apparently anomalous behavior of the chloride and sulfate ions might be due, not to these anions, but to the greater concentration of sodium ion furnished by the sodium chloride. If this is true, a bivalent cation should exert a greater effect than the monovalent sodium cation at pH-values above 6.5.

In order to test the theory, two series of experiments were made with ferric sulfate as coagulant. In one set chloride ion was supplied in the

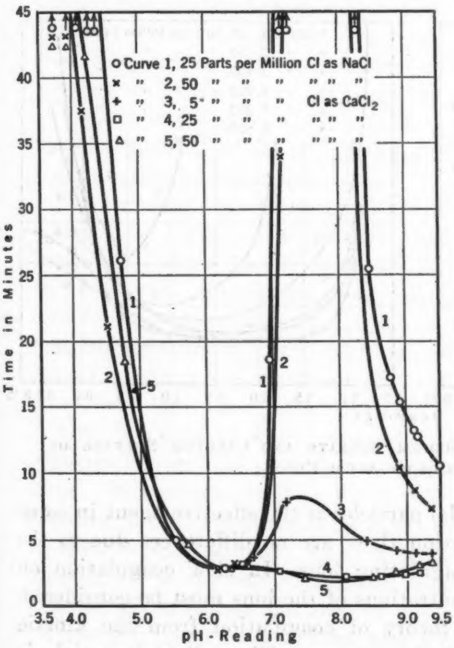


FIG. 7.—COMPARISON OF EFFECT OF SODIUM CHLORIDE AND CALCIUM CHLORIDE ON FLOC FORMATION WITH FERRIC SULFATE.

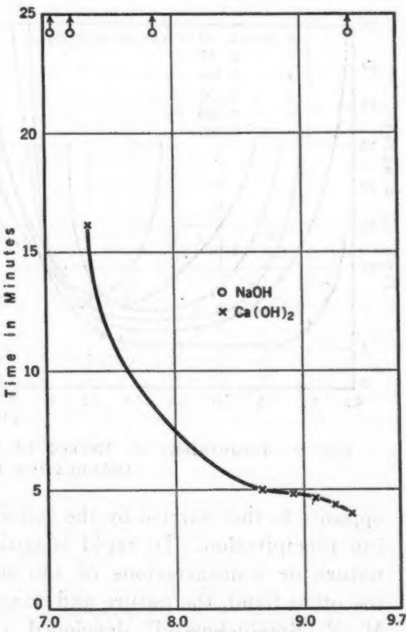


FIG. 8.—COMPARISON OF SODIUM AND CALCIUM IONS ON FLOC FORMATION WITH FERRIC SULPHATE.

form of sodium chloride and in the other, in an equal amount, in the form of calcium chloride. The results obtained furnish strong evidence for the correctness of the theory. Flocs formed more quickly in solutions containing only 5 ppm of chloride from calcium chloride than in solutions containing 50 ppm of chloride from sodium chloride. The solutions containing 25 to 50 ppm of chloride from calcium chloride formed more quickly than any other flocs worked with in the entire investigation. (See Fig. 7.) To test the theory further two series of experiments were made using only distilled water and coagulant, supplying the alkali in one series with sodium hydroxide and in

the other series with calcium hydroxide. Where sodium hydroxide was used, no floc formed in 45 min at pH-values between 7 and 9.0, whereas when calcium hydroxide was used, a floc formed in 16 min at a pH-value of 7.3, and in 5 min between pH-values of 8.6 and 9.4. (See Fig. 8.)

Messrs. Bartow and Peterson,[4] in one set of experiments, made a comparison of the relative effects of equal dosages of sodium and calcium sulfates on the time of formation of alum floc. Their results (see Fig. 9) reveal the interesting fact that, while the differences are not marked, calcium sulfate gave wider zones of floc formation on the alkaline side than were given by sodium sulfate, used in equivalent amounts.

Colloid chemists make a distinction between the rapid coagulation and the slow coagulation of hydrophobic sols. In both cases the ion of charge

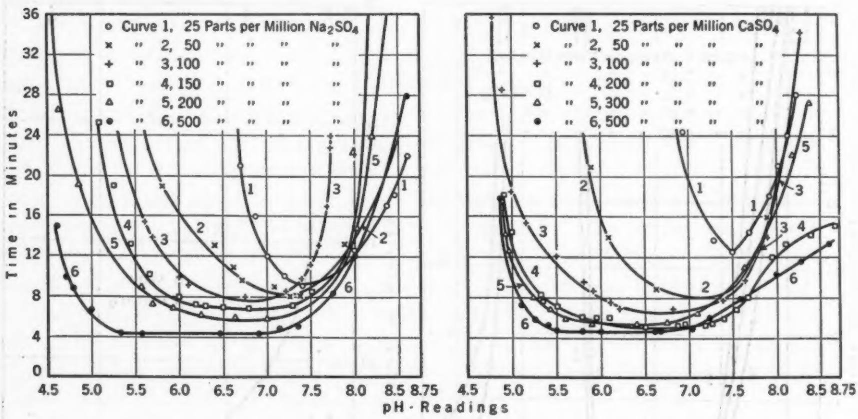


FIG. 9.—COMPARISON OF EFFECT OF SODIUM SULFATE AND CALCIUM SULFATE ON COAGULATION TIME OF ALUM FLOC.

opposite to that carried by the colloidal particles is the effective agent in causing precipitation. In rapid coagulation there are no differences due to the nature or concentrations of the coagulating ions. In slow coagulation on the other hand, the nature and concentrations of the ions must be considered. M. V. Smoluchowski[25] developed a theory of coagulation from the kinetic point of view, which is confirmed by observations. The colloidal particle is assumed to consist of a central charged particle surrounded by an envelope of ions of opposite charge. This envelope of ions may extend rather deeply into the liquid phase, and the double layer so produced gives rise to a potential difference termed the ζ-potential. The repulsive forces between the sol particles are due to this ζ-potential and in the range of rapid coagulation it is abolished. He developed the following formula:

$$K_r = 4\pi D l \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, K_r is the velocity constant, assuming that the coagulation takes place as a dimolecular reaction: D, the diffusion constant of the particles;

[25] *Zeitschrift für Physikalische Chemie*, Vol. 92 (1917), p. 155.

and l, the distance from each other to which two particles must approach, in order that the collision between them becomes effective. There is no factor introducing the nature and concentration of the electrolyte producing coagulation.

For slow coagulation, Mr. Smoluchowski[25] introduces an additional factor into the equation. In this case, the formula is stated:

$$K_s = 4\pi D l \xi \qquad (2)$$

in which, ξ signifies that only a certain fraction, ξ, of all collisions results in coagulation. This fraction, ξ, depends very markedly upon the nature and concentration of the electrolyte. The ion of charge similar to that of the ionic envelope tends to diminish the depth of the envelope, the resulting distortion giving rise to attractive forces. The ion may also be adsorbed on the surface of the particle or even react chemically with it. In the zone of slow coagulation, only a certain fraction of the collisions is effective, due possibly to the fact that the particles do not possess sufficient kinetic energy to overcome the ζ-potential. If it is assumed that the distribution of the kinetic velocities of the sol particles follows Maxwell's curve, then the results are as would be expected.

In terms of this theory, the behavior of the ferric hydroxide floc might possibly be explained as follows: On the acid side the colloid consists of positively charged particles of ferric hydroxide plus basic ferric sulfate or ferric chloride, surrounded by an envelope of sulfate or chloride ions. In the zone of slow coagulation on the acid side, the sulfate ion is far more effective than the chloride ion and both are effective in proportion to their concentrations. At a pH of about 6.5, at which point rapid coagulation takes place, both ions are equally effective. On the alkaline side of this point there is a second zone of slow coagulation in which the colloidal particles bear a negative charge due possibly to adsorption of hydroxide ions from the solution. In this zone (pH = 6.5 to 8.5), the bivalent calcium ion is far more effective than the monovalent sodium ion, and both are effective in proportion to their concentrations. At still higher pH-values, a second zone of rapid coagulation is approached in which zone both are equally effective. In order to establish definitely the sign of the charge carried by the colloidal floc at various pH-values on both sides of the neutral point, cataphoresis experiments have been begun.

If further work should confirm this apparent change in the sign of the colloidal floc on the alkaline side. it might serve to explain in part the difficulty that is often experienced in color removal in this pH-range, 6.5 to 8.5.

Color is usually present in natural waters in the form of a negative colloid, and it has been assumed that it is sufficiently acid to be appreciably soluble at the higher alkaline pH-values often used in coagulation. While this seems well established, yet the difficulties inherent in the removal of a color colloid by a colloidal floc of the same sign are obvious.

Residual Iron in Solution at Varying pH-Values.—Experiments were made to determine the quantity of iron remaining in solution at various pH-

values. A series of runs was made in the usual manner with chlorinated copperas as coagulant. Sulfate ion (25 ppm) was added in the form of sodium sulfate and the hydrogen-ion concentration adjusted with sodium hydroxide at various pH-values from 3.5 to 9.4. The iron remaining in the solution (see Fig. 10), at various pH-values, correlates with the times of floc formation. There is a wide range on the acid side in which little or no residual iron is present. Beyond pH 9.0, when the floc again begins to form,

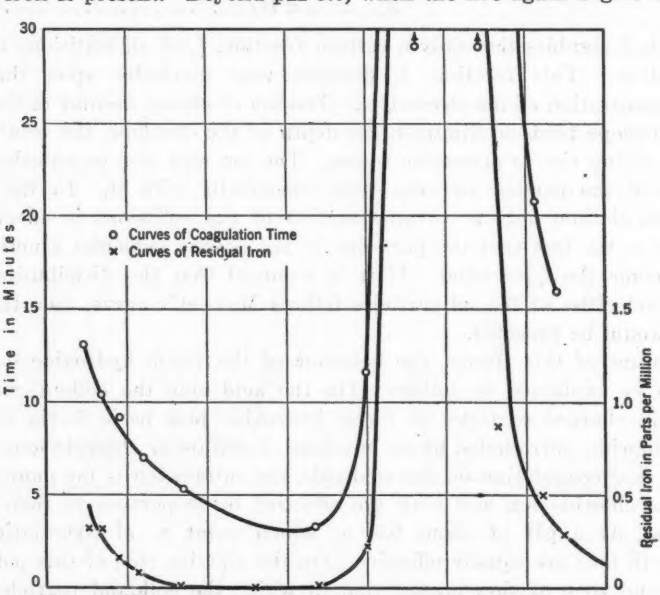


FIG. 10.—RESIDUAL IRON FROM FORMATION OF FERRIC FLOC AT VARYING pH-VALUES, USING 1.6 GRAINS OF CHLORINATED COPPERAS AND 25 PARTS PER MILLION ADDITIONAL SULFATE.

the quantities of residual iron decrease rapidly with decreasing time of floc formation. Determinations were not made beyond pH 9.4 since Mr. Hopkins[21] has shown that the floc is highly insoluble in the pH-range, 9.4 to 13.0. The very high values which he obtained at lower pH-values (11.0 ppm at pH 6.0) are undoubtedly due not to solubility of the ferric hydroxide floc, but to the incomplete oxidation of the ferrous sulfate that he used as coagulant.

No attempt has been made to investigate the chemical composition of the floc formed at various pH-values, since Messrs. Miller[14] and Hopkins[21] showed rather definitely that the floc is a basic sulfate or chloride until three equivalents of alkali have been added. The molecular ratios, $\frac{SO_4}{Fe}$ and $\frac{Cl}{Fe}$, increase as the pH decreases. When three equivalents of alkali are present, the floc may be washed free of sulfate or chloride and appears to be true hydrated ferric oxide.

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PAPERS

MODIFYING THE PHYSIOGRAPHICAL BALANCE BY CONSERVATION MEASURES

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Synopsis

In Nature, precipitation, erosion, débris transportation and deposition by stream flow combine to produce, in some measure, a physiographical balance which, while not permanent, undergoes only gradual changes. This balance is expressed in the topography and cover of the mountain water-shed as well as in that of the flood-plain of a drainage area.

Interference with the natural processes of erosion, débris transportation, and disposal of flood waters of the stream of a water-shed by partial or complete regulation, may result in a modification of this physiographical balance to such an extent as to menace existing or future development. This modification of balance may be a slow process, apparent only after a considerable period of time and, for this reason, may be overlooked or considered as negligible; yet it may prove to be the undoing of otherwise well-laid plans.

The effect of interference with natural conditions is accentuated in the semi-arid regions because of the greater extremes of precipitation and other climatic factors which tend toward the accumulation of débris during dry periods. In the Southwestern United States, the regulation of water supply and the control of flood waters has brought about the erection of works of unprecedented size, which regulate large floods and permit the artificial disposal of the entire stream flow.

This paper is intended to call attention to the effect, on the physiographical balance, of the following conservation measures: (*a*) Changes in the water-shed cover; (*b*) effect of regulation on the natural balance of a stream system; and (*c*) effect of débris barriers on the stability of the stream bed on the débris cone.

Note.—Discussion on this paper will be closed in March, 1934, *Proceedings*.

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In the examples presented, it was not so much the intention to discuss in great detail the phenomena which are affected by the modifying of the physiographical balance, but rather to illustrate the theory. For this reason, the problems arising in the mountain water-shed and those which arise in the transportation of débris are not discussed exhaustively.

1.—The Physiographical Balance

The physiographical appearance of a country is generally the combined result of orogenic (mountain-forming) movement and climatic conditions. Crustal movement is responsible for the basic physical features of a country. This basic relief is, however, gradually and continuously modified by climatic factors.

Of the climatic factors, precipitation and the processes accompanying its disposal, are principally responsible for the existing aspects of a country. Although the distribution of precipitation may vary in intensity with seasons and periods, past performance indicates, nevertheless, that over long periods its general characteristics and effects may be adjusted to a mean of conditions governing changes in terrestrial appearance which, to the human eye, is seemingly more or less permanent and unchanging. In other words, despite their erratic character, climatic factors in general, and precipitation in particular, produce in Nature what might be termed a state of equilibrium, or balance, expressed in a certain type and density of native vegetation, in more or less stable gradients of stream beds, in the disposal of débris, in the slopes of the plains, etc. It is a balance of natural forces which is seemingly maintained while, at the same time, the phenomenon of stream erosion gradually produces changes on land surfaces.

Any long-sustained change in the mean precipitation would naturally be followed by an adjustment to new conditions. A decrease in rainfall of 20% may transform habitable semi-arid areas into uninhabitable deserts. An increase in the mean precipitation by the same amount would result in less severe droughts, increased mean water supply, and in flood conditions of much greater severity. The vegetal cover of the water-shed would adapt itself gradually to the more abundant supply, becoming more profuse, but possibly less drought-resistant. Changed run-off conditions would gradually alter the regimen of the watercourses, including their gradients, stream beds, and character and volume of débris. Slopes of the plains, too, would be affected as well as the vegetation thereon; and ground-waters would rise above their former levels. In other words, the face of the country would alter gradually, but inevitably, until the new balance of geophysical forces becomes seemingly established, in conformity with the changed conditions.

The effect on human life of a long-sustained, substantial increase in wetness might be as far-reaching as that of greater prevalence of drought. In fact, it is conceivable that the disturbances during the period of transition may make human habitation in specific localities difficult, if not impossible, to be maintained.

If, then, there is a natural balance of physical effects, as the result of ages of adjustment to existing conditions, it is logical to infer that any interference therewith by human agencies is likely to result in disturbances. Those disturbances may, or may not, become apparent immediately, but are sure to follow and to remain as long as the cause exists. The very fact that such results are not immediate, or become felt only gradually, may be the reason why they are sometimes overlooked or under-estimated until they have assumed proportions that require heroic counter-measures, or may even have passed beyond control—comparable to the experience of the sorcerer's apprentice who conjured the spirits of witchcraft, but was unable to dismiss them.

A simple, every-day occurrence may serve to explain the continued and relatively far-reaching effect of interference with Nature's equilibrium. Under natural conditions, the gently sloping sides of the foothills are covered with certain types of vegetation which prevent the washing of soil and which are inducive to absorption of rainfall, producing a normal condition preventive of flood run-off, thus affording a physiographical balance on these foothills. If these slopes are cleared and cultivated, erosive action will immediately be noticeable, even if protection by cover crops is provided. It is not uncommon, therefore, that after some years the plow may strike an irrigation pipe line which originally had been buried the customary 2 ft under the surface.

Precipitation causes innumerable rivulets and barrancas on such slopes; flood waste is increased, while deep penetration is diminished, contrary to the opinion sometimes advanced that plowing favors larger absorption of rainfall. Where such slopes are devoted to residential settlement, rainfall penetration will be diminished further, augmenting erosion and flood waste to such proportions as to necessitate costly terracing and the construction of lined storm channels. The one effect which may not be immediately apparent, but may be the most far-reaching, in the long run, is accelerated erosion.

It is concluded that the physiographical condition of an area is the result of the activity and balance of natural processes of precipitation, erosion, and débris transportation and deposition by stream flow over a long period. A pronounced or protracted modification of these phenomena will initiate changes tending to establish a different physiographic balance. Although these changes may appear to be gradual and negligible, their effect, nevertheless, is irresistible and in due time will upset existing conditions.

2.—Changes in the Water-Shed Cover and Their Effect on Run-Off Conditions

Plans for the "improvement" of the water-shed for the purpose of increasing the water yield have been advanced of late years from many quarters. Radical changes in the water-shed cover have a more or less protracted effect on débris production, run-off, and the capacity of the stream to move its load. Aside from the timely agitation for the reforestation of cut-over or

burnt-over timber lands, the proposals may generally be segregated into two classes:

First.—"Super-forestation"[2] to the effect of producing a denser growth of native or other vegetation on the water-shed; to "keep the water supply in the water-shed," as the saying goes; and to induce more copious precipitation.

Second.—"Light deforestation" to restrict the vegetation to a minimum compatible with effective prevention of erosion, with a view to reducing the evapo-transpiration requirements of the cover.

Super-forestation (and forestation for that matter) is analyzed as an effort to increase the storage of meteoric water in the water-shed by promoting profuse vegetation which naturally tends to absorb rainfall and diminish flood run-off. By this means surface soils may be better protected from washing, and débris production may be reduced. However, a dense vegetation may also be the cause of more intense forest fires and greater difficulties pertaining to fire protection and fire-fighting. Indirectly, the more complete destruction of plant life by fire, as a result of super-forestation, may be the cause of retarding re-growth and exposing the water-shed to a longer period of abnormal erosion.

Under the caption of reforestation comes the proposal of replacing chaparral by trees. However, since chaparral recovers naturally and more rapidly from fire than trees which would have to be replanted, the effect on the water-shed cover would be decidedly disadvantageous and would be most likely to protract the period of abnormal erosion after a fire.

Efforts at "super-forestation" on a large scale are not likely to meet with success. The general type of the forest cover of a region is indigenous to it and reflects its climatical conditions—mean temperatures, mean precipitation, and other climatic factors—while the range in precipitation and temperatures is reflected in the tolerance of the species, all to the effect that Nature has produced the vegetal cover which, both as to species and density, is best adapted to prevailing conditions. Any attempt, therefore, to induce denser growth is likely to fail, due to the great "weeding-out" process expressed in the law of the survival of the fittest.

Light deforestation is to be considered an effort to reduce underground storage in the water-shed and the evapo-transpiration losses of the cover, at the expense, however, of a more rapid, flashy, surface run-off and correspondingly increased erosive action. Under this category may be grouped: (*a*) The systematic burning of the water-shed; (*b*) the substitution of light water-using species for heavy water users; and (*c*) the cutting of trees which grow along the watercourses, such as alders, willows, sycamores, and the like. Similar results as to light deforestation would result from a diminished vigilance relative to forest fires.

The practicability of these proposals has never been put to a test. The most far-reaching plan in its effect on conditions of stream flow and débris production, and the one entailing the greatest risk, would be the systematic

[2] "Forests and Stream Flow," by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., November, 1932, p. 1615.

and periodic burning of the chaparral. This has been under discussion and advocated in some parts of California—first, with a view of increasing the water supply; and, second, to prevent the accumulation, over long periods, of masses of highly inflammable débris which results under the existing official policy of protection of the water-shed from fire. Burning is proposed to be done in checker-board fashion at the end of the wet season when humidity is relatively high and when soil moisture is sufficient to produce new sprouts of brush, grasses, and herbs.

This proposal and others is of a highly controversial character. Although advocated in certain localities with considerable popular and political pressure, those in charge of water-sheds and responsible for damages, should the burning get beyond control, have been slow in yielding to the demand. In a report of the Ventura County (California) Farm Bureau[3] relative to the burning, it is stated:

> "The general consensus of opinion among those with great experience in fire-fighting is that when it is dry enough to burn, it is too dangerous to burn, and when it is safe to burn, the vegetation will not burn."

The effect of denudation by fire manifested in flashy, heavy flood flow and abnormal débris load, the possible destruction of surface storage, and the flooding of the plains, is too well understood to require discussion. A point sometimes overlooked is the sealing effect of ashes on the stream bed, which greatly reduces natural seepage, to the extent of more than offsetting the increase in the water yield of the burnt-over water-shed.

However, forest fires will be a factor as long as the public is allowed free access to the mountain areas. They will have to be reckoned with to an increasing degree in the planning of flood-control and conservation measures.

The forest cover unquestionably has an important constructive function in Nature, which may not be fully appreciated because it is not well understood. Its preservation should be uppermost in the minds of water conservationists because of the far-reaching and lasting effects that destruction or denudation of the water-shed will have on flood flow and on the production and movement of débris. It is not generally recognized that domestic water supply, irrigation, and hydro-electric enterprise have rights in the control of the water-shed. In settled regions, the value of the water-shed for natural storage and regulation of run-off may be superior to any other uses, such as grazing, recreation, or even the commercial exploitation of its timber and, therefore, should receive due consideration.

In the past there has been little or no co-operation between the forester, the engineer, and the water user, except to the extent of protection of the water-shed from fire hazard. A step toward a better understanding of the common problem was undertaken by the Committee on Water Conservation of the Irrigation Division of the Society, by calling a conference[4] of

[3] Mimeographed Rept., Conservation Committee, Ventura County Farm Bureau, April 8, 1931.

[4] Mimeographed Rept. on "Conference of Research Problems in Consumptive Use of Water and Conservation of Rainfall, Los Angeles, Calif., March 27, 28, and 29, 1930."

research workers, engineers, foresters, and agriculturists. This meeting resulted in the outlining of a program to correlate the work of the various groups relative to conservation of water supply.

As yet, the function of the water-shed in its ramifications is not well understood. The United States Forest Service in late years has initiated a comprehensive program of research with a view of increasing the water yield and decreasing the fire hazard. When these investigations have progressed to the point where definite conclusions can be drawn as to the practicability of the proposed plans and their ultimate effect on the water yield, a logical solution may present itself. From the standpoint of general economics and efficiency of flood control and conservation, proper co-ordination of the improvements of the water-shed to those of the flood channel becomes a necessity.

3.—Effect of Regulation Upon the Natural Balance of a Stream System

The work performed by a stream or stream system may be segregated into three distinct functions: First, erosive action on the mountain water-shed; second, the transportation of débris; and, third, the building of débris slopes, or flood-plains.

With undisturbed conditions, each of these activities depends directly upon the combined effect of stream flow and available débris. Since both stream flow and volume of débris are subject to great and, sometimes, erratic fluctuations, it becomes apparent that the magnitude of these uncontrolled forces is forever variable, without attaining uniformity for any great length of time. However, available records indicate that, considering cycles of several years, these stream activities occur between limits that are defined by droughts and capital floods, so that in the intervals between such extremes there is a constant tendency to 'iron out" the effects of individual floods or periods of drought. There results a state of balance which is characteristic for a stream, or stream system, termed its "regimen." It finds expression in the gradient of the flood-plain and the character of its débris and alluvium.

Stream activity follows lines of least resistance and, therefore, will tend to establish the steepest gradient consistent with the basic factors—stream flow and débris. Only long-sustained climatic changes or the exposure of different geological formations, or a change in botanical cover over large areas of the water-shed, would materially affect the normal gradients and change the character of the débris of the flood-plain. The characteristics of a stream system, therefore, can be established by a study of its deposits and of its gradients.

Generally speaking, and in comparison with the time allotted individual human experience, water-shed erosion and the construction of the flood-plain are relatively slow, but ceaseless, processes.

Nevertheless, if, as a result of the construction of an impounding reservoir, the transportation of the débris is interrupted for a period of years, or if it is stopped, a permanent effect on the stream bed and flood-plain below is certain to follow.

If the stream bed is used for the diversion of the impounded water, the clarified effluent—having the capacity to load itself anew with sediment—will begin to scour and meander, reducing its gradient and changing its wetted perimeter. These new conditions, acting on the alluvium of the former natural stream, therefore, will establish a new state of equilibrium, in due time, expressed in flatter gradients. A further and vital change will occur at the point below the reservoir where the stream intercepts the first major tributary.

With a stream system functioning normally, there is a balance of forces between the flow of the main stream, its gradients, and the débris carried by it and those similar functions of its tributaries. The addition of a tributary brings about a flatter gradient in the main stream for some distance above the junction, while at and below the junction such steeper slope is established as may be required for the movement of the added débris. The tributary, as a rule, has a lesser run-off and steeper gradient than the main stream, its characteristics being reflected in its own flood-plain where it has established the most favorable conditions for the transportation of its own débris under its own conditions of flow.

If the main stream is regulated while the tributary remains unregulated, sooner or later, conditions at the junction of the two will become critical because of the absence of the flood flow in the main stream, which formerly was available for the removal of the sediments and débris from the tributary. The tributary, now having to assume the rôle of the main stream, cannot cope with the wider stream bed and flatter grade of the latter, hence the deposition of sediments, a raising of both stream beds, and inundations at the junction.

A natural correction of these conditions cannot take place unless storage is released at intervals and in sufficient quantities to cause the movement of the excess sediment. However, since this remedy may not prove effective without serious infringement on storage, the only other solutions will be: (*a*) To improve conditions of flow in the main stream below the reservoir by straightening the alignment, to line the channel to meet the flow requirements of the tributary, or both; (*b*) to regulate the tributary itself by means of surface storage, as in the case of the main stream; and (*c*) to construct débris barriers on the tributary for the purpose of reducing its load.

In any event, if tributaries are of sufficient magnitude, the problem may prove to be a baffling one and not easily solved. If it can be solved at all, it may require the expenditure of relatively large sums, or the periodical removal of débris.

The depth of deposits when spread over a wide flood-plain even during a period of years, is relatively insignificant and its absence would not be noticeable or of material effect in a life time. From the geological standpoint, the volume of débris stored in the largest reservoirs is small.

Some observations pertinent to the foregoing discussion have been made in Southern California. During a series of dry years, the San Gabriel River with 218 sq miles of drainage area may have little or no flood run-off, because

all the normal flow has been diverted, so that the stream bed remains dry over long distances below the mouth of the canyon. During the same period, the tributaries flowing in more or less trained channels and receiving the run-off from urban areas, have high, flashy silt-laden flows with every rain. At the point of junction with the river, this silt is deposited in the river bed, filling it enough so that inundation of adjacent lands during floods becomes a possibility.

In the Western States, the regulation, and, in some cases, the ultimate complete diversion of the flow, of large rivers form the bases of many reclamation and flood-control projects.

The regulation of a stream presents one problem and the effect of this regulation upon the tributaries becomes the cause of another problem. Although this effect may be gradual and over a period of normal stream flow create no menace, a new condition has been initiated which must be met sooner or later. In planning regulation works, this phase should receive the engineer's attention, as affecting either the quantity of storage that may become available for beneficial use, or requiring measures for the improvement of tributaries. The question arises in particular, as to whether it is feasible to regulate the capital flood run-off for irrigation or domestic use. In some cases this has been advocated regardless of costs when such flood flow might be wasted more profitably for the removal of débris.

Experience on the Rio Grande, in Texas, as a result of the construction of the Elephant Butte Dam, will serve to illustrate the foregoing analysis. In that case the problem is complicated by the use of the river channel below the reservoir for the conveyance of irrigation supplies.

4.—The Effect of Regulation of Rio Grande on Flood Conditions Below Elephant Butte Dam

The Rio Grande drainage basin above El Paso, Tex., occupies the south central part of Colorado and the central part of New Mexico. Drainage areas are divided as follows: Above San Marcial, N. Mex., 26 700 sq miles; above Elephant Butte Dam, 28 360 sq miles; and above El Paso, 30 460 sq miles. Fig. 1 shows the principal geographic and topographic features of Rio Grande Valley from San Marcial (Elevation 4 458), at the north end of Elephant Butte Reservoir, to Fabens, Tex. (Elevation 3 619) on the Rio Grande, 26 miles below El Paso.

Elephant Butte Dam is located across the bed of the stream about 125 miles above El Paso, and creates a reservoir of about 2 600 000 acre-ft capacity. The mean annual run-off at San Marcial, at the head of this reservoir, is roughly 1 200 000 acre-ft. The entire discharge of the Rio Grande has been retained in the reservoir (except as released for irrigation use) since storage operations started in 1915. The highest recorded discharge at San Marcial prior to 1930, was 33 000 sec-ft and the highest passing El Paso prior to the construction of Elephant Butte Dam was 24 000 sec-ft. The highest discharge at El Paso since the construction of Elephant Butte Dam is 13 500 sec-ft. The large floods which occurred at intervals of several

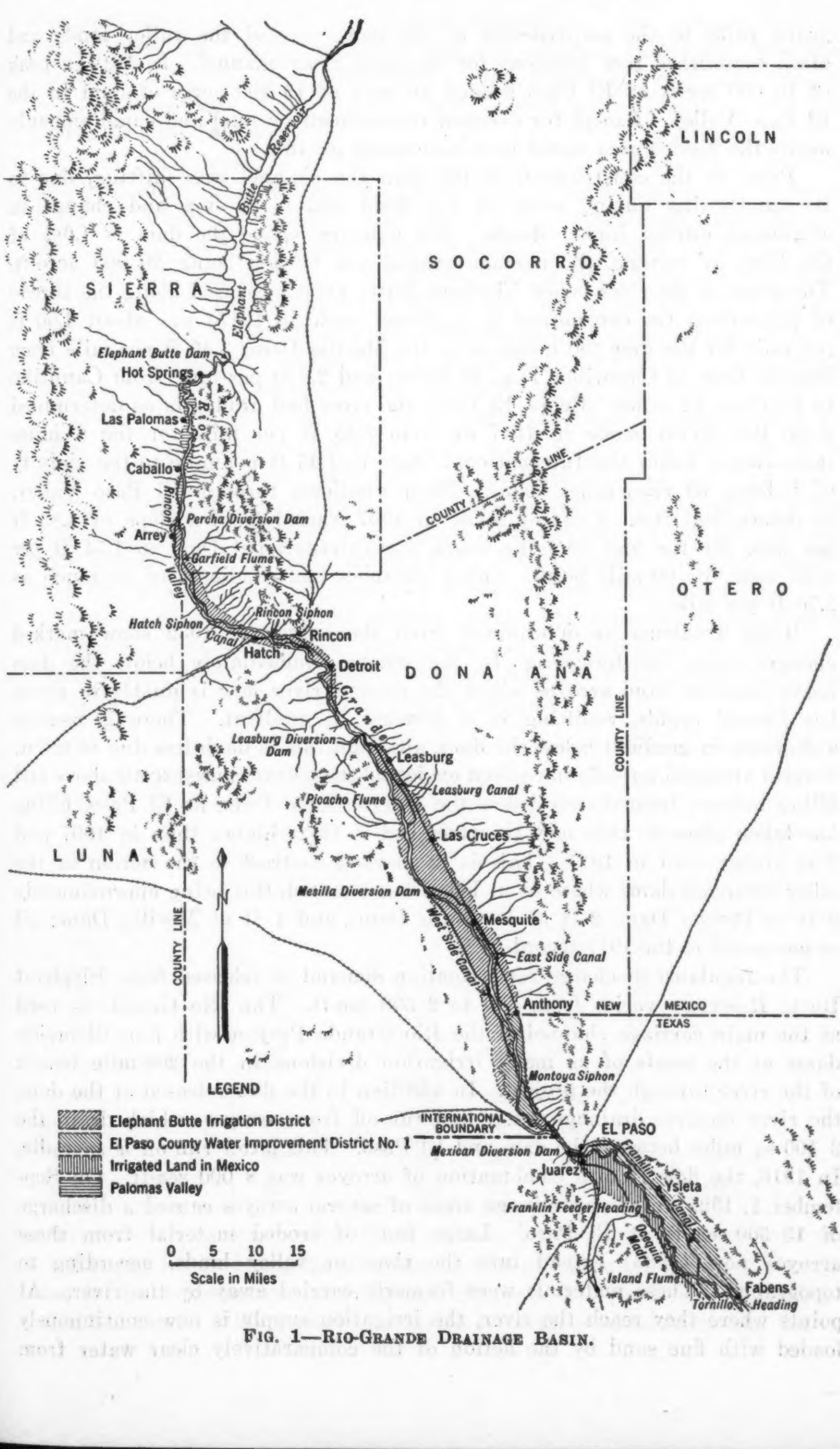

FIG. 1—RIO-GRANDE DRAINAGE BASIN.

years, prior to the construction of the dam, covered the valley lands and often established new locations for the main river channel. In 1912, a peak of 16 000 sec-ft at El Paso flooded an area of 15 900 acres of land in the El Paso Valley. Except for overflow confinement by road and canal embankments the flooded area would have been much greater.

Prior to the construction of the dam the channel was shifting, due to its lengthening during years of low flood and to erosion and change in alignment during higher floods. Silt entering above the dam is 1.6% of the flow, by volume, the average annual silt burden being 20 000 acre-ft. The slope of the river below Elephant Butte Dam measured along the thread of the stream (as determined by a survey made in 1917) was about 4.00 ft per mile for the first 100 miles, or to the Mesilla Dam; 3.50 ft per mile from Mesilla Dam to Canutillo, Tex., 30 miles; and 2.8 ft per mile from Canutillo to El Paso, 15 miles. Below El Paso, the river had gradients as determined from the survey made in 1917 of from 2.45 ft per mile for the 6 miles immediately below the International Dam, to 1.45 ft per mile to the vicinity of Fabens, 50 river-miles below. River gradients through El Paso Valley, as determined from a survey made in 1907, varied from a slope of 1.83 ft per mile for the first 20 miles below the International Dam to 1.54 ft per mile near the 60-mile point. Other stretches had gradients of as much as 2.70 ft per mile.

River gradients as determined from the survey of 1932 show marked changes from the foregoing. In the reaches immediately below the dam heavy detritus from arroyos which the present river flow is unable to move, has formed rapids, resulting in a decrease in gradient. There is usually a decrease in gradient below the dam, although this is doubtless due to scour. Several artificial cut-offs have been made and these have caused scour above and filling below. Immediately below the International Dam, at El Paso, filling has taken place so that now the river bed is 12 ft higher than in 1907 and 6 ft higher than in 1917. This is in marked contrast to the action at the other diversion dams where scour below has resulted, this being approximately 6 ft at Percha Dam, 2 ft at Leasburg Dam; and 1 ft at Mesilla Dam; all as compared to the 1917 record.

The regulated discharge for irrigation demand as released from Elephant Butte Reservoir varies from 500 to 2 500 sec-ft. The Rio Grande is used as the main carriage channel of the Rio Grande Project with four diversion dams at the heads of as many irrigation divisions in the 200-mile length of the river through the project. In addition to the flow released at the dam, the river receives drainage water and run-off from arroyos, which drain the 2 100 sq miles between the dam and El Paso. This latter run-off is sporadic. In 1916, the flow from a combination of arroyos was 8 000 sec-ft. On September 1, 1925, rains on drainage areas of several arroyos caused a discharge of 13 500 sec-ft at El Paso. Large fans of eroded material from these arroyos now (1933) extend into the river or valley lands, according to topography. These materials were formerly carried away by the river. At points where they reach the river, the irrigation supply is now continuously loaded with fine sand by the action of the comparatively clear water from

Elephant Butte Reservoir on these accumulations. The concentrated discharge from one arroyo, resulting from an isolated cloudburst, has often equalled the discharge of all arroyos during a general rain. The drainage areas of these arroyos varying from a few square miles to 400 sq miles, are partly denuded of growth.

With the completion of the storage works, and the assurance of water supply throughout the irrigation season, the irrigated area was enlarged. Drainage canal embankments and lateral canals constructed near the river channel confined the flow to a more definite location and the deposition of sediment to a limited area. The former meandering river channel became the main canal for the project. Maximum flows were reduced more than 50%, the mean annual flows throughout the project were reduced 30%, and the run-off extended throughout the length of the irrigation season of nine months, rather than being concentrated in the spring or fall.

Project diversion works, as well as principal canal structures, were provided with skimming weirs and canal wasteways to the river, which are constructed and operated to return the sand sediment to the river and to prevent large maintenance costs. As these desilting devices were perfected and general sluicing operations became more effective, sand accumulations were concentrated in the river channel. River profiles show the effect of such deposits at diversion points and at the outlets of the wasteways.

Cross-sections of the river channel before and after construction of the storage unit are of interest in expressing the actual effect of the control and regulation of discharge on the former river channel. Table 1 gives the change in areas of cross-sections below a given elevation.

TABLE 1.—CROSS-SECTIONS OF RIO GRANDE BELOW ELEPHANT BUTTE DAM
(Areas, in Square Feet, Below a Given Elevation)

Section	Miles below Elephant Butte Dam	1915	1918	1932	Difference	Bottom		Side	
						Scour	Fill	Erosion	Fill
Hot Springs, N. Mex....	7	4 970	.....	4 671	—299	626	.....	.....	925
Garfield, N. Mex........	36	.....	1 528	2 228	+700	700	.....	.....	.....
Hatch, N. Mex..........	54	.....	1 669	2 066	+397	579	.....	.....	182
Picacho, N. Mex........	87	738	.....	1 178	+440	440	.....	.....	.....
Line 34..................	93	.....	5 359	4 301	—1 058	449	.....	.....	1 507
Line 30..................	97	.....	9 568	8 393	—1 175	.....	1 175	.....	.....
Line 25..................	105	.....	1 642	1 928	+286	286	.....	.....	.....
Line 13..................	120	.....	1 410	1 812	+402	356	.....	46	.....
Line 5...................	129	.....	1 663	2 066	+403	414	.....	.....	11
International Dam.....	143	.....	.....	.....	......	......	.....	.....	.....
Colorado................	144	.....	(1929) 2 213	1 769	—444	.....	448	4	.....
Socorro, N. Mex........	168	.....	(1927) 320	162	—158	.....	158	.....	.....
Flume....................	184	.....	(1920) 1 548	1 270	—278	.....	278	.....	.....
Line 61..................	236	.....	(1926) 696	443	—253	.....	218	.....	35
Line 65..................	267	.....	679	519	—160	.....	140	.....	20

As may be expected, the decrease in flow at the lower end of the project, due to the consumption for irrigation purposes, has resulted in a closing in of the channel. This is shown in Table 1 for all sections below the Interna-

tional Dam. Furthermore, side fill is shown in many of the sections and immediately above the Mesilla Dam, "Line 30," bottom fill has resulted due probably to the checking effect at that dam. The absence of large destructive floods has afforded no opportunity for the former scouring action, and the river channel below El Paso has filled with sand sediment to a point where it is higher in many places than the adjoining farm area. As the channel thus conforms itself to the normal irrigation flow, floods from excess rainfall on the drainage area below the dam cause the overflow of the low-lying areas adjoining the river channel. The elevation of the river channel, of course, is no new condition. Practically all the valley lands were formed by river deposits of silt and sand from upper drainage areas.

Thus, the outstanding characteristics of the present regimen of the Rio Grande are: (1) The entire absence of large floods to scour the bottom of the river bed; (2) continuous flow in the river of a quantity sufficient to cover the necessities of irrigation, a flow that produces a continuous accumulation of sediment which is deposited in that part of the valley below El Paso, where the decrease of the flow and the reduction of the slope favor this deposit; (3) diminution of the flood area of the river in the lower parts of the valley, in consequence of this continuous deposit of sediments; (4) a river bed at present inadequate to permit the free passage of the waste waters of floods that may rise from the combination of the simultaneous discharge into the river from the various tributary arroyos below Elephant Butte Dam; and (5) greater actual danger of inundation of the lower river lands caused by the waste waters of the present smaller floods than those that occurred before the construction of the Elephant Butte Dam.

The proposed solution of the problem consists of shortening the river from 155 miles to 88 miles between El Paso and Quitman Canyon, by making cuts across bends. In this manner the slope of the channel is changed from 1.8 ft to 3.2 ft per mile.

As the silt content of the river at El Paso is only about 500 acre-ft per year, it is anticipated that following this rectification of the main channel, the flood and normal irrigation flow of tributaries will be of sufficient magnitude to produce scouring and to establish a new stable channel.

5.—Effect of Débris Barriers on the Stability of the Stream Bed on the Débris Cone

The stabilizing of stream courses on the cone has become a factor of importance in the semi-arid areas of the Southwest because the hillside and the cone have been proved to enjoy better protection from frosts than valley areas and, hence, to be better adapted to high-class residential development and to the culture of semi-tropical fruits. Furthermore, the availability of deep gravel deposits and the general commanding elevation above irrigable areas make the cone the most desirable location for the conservation of flood waste by water-spreading in contour ditches or ponds. Hence, the necessity of diverting flood waters on the cone by means of diversion dams into canals and ditches and of confining flood areas by levees for the protection of spreading grounds and residential areas.

Stream Action on the Cone.—The débris cone of a stream is defined herein as the alluvial deposit accumulated below the mouth of its canyon and over which the stream periodically meanders. When the flood stream emerges from the confining walls of the canyon, the path is clear for a lateral spreading. Velocities, therefore, will decrease and so will the transporting power of the stream; all to the effect that sediments and detritus will be deposited on the cone, more or less graded from the coarsest at the apex to fine materials at its base. This alluvial action constantly obstructs and tends to force the stream into other paths. The location of the stream bed on the cone is, therefore, inherently unstable, because there is no longer a balance between stream flow and the débris in traction.

As a rule, streams arrive at the base of the cone in broad sheets, which, having delivered themselves of their burden, then again collect into one, or possibly more, channels. As this concentration of flow takes place, velocities increase, scouring activity is again initiated, and, eventually, a single stream bed in cut is produced. Thus, a cycle of events is established: Erosive action in the mountain water-shed, the deposition of the débris on the cone, and the issuance from the cone of a stream delivered of its normal load and again capable of scouring its bed.

The apex of the cone being at the mouth of the canyon, shifting of the stream bed is likely to follow more or less along lines issuing radially from the apex, so that any structure or improvement on the cone is exposed to the danger of being flooded, washed out, or covered with débris. Little discussion is needed to show that any attempt to establish a balance on the cone between stream flow and débris by confining the stream between levees, can only be temporarily successful or, at best, will merely shift the location of deposit to lower areas.

It is generally accepted that a riparian owner may lawfully protect his property from the ravages of floods by means of levees, while diversion or other interference with natural conditions of stream flow is at the manipulator's risk. Considering then the instability of a stream on the cone, its manipulation will involve responsibilities which may assume unexpected proportions.

However, there are localities (and they are numerous in California) in which cones are among the choicest frost-free foothill lands, suitable for high-class agriculture or residences. Large areas have thus been improved, representing investments of thousands of dollars per acre and owners, therefore, demand protection from potential hazards, whether caused from natural or from artificial features.

Débris Barriers.—Débris deposition being the basic cause of the instability of stream courses on the cone, the apparent remedy is storage of the debris in the canyon above. A stream thus delivered of its burden has increased capacity for erosive action below, picking up its "normal load" on its course down the cone. In this manner, it may reverse its action from alluvial to erosive and automatically stabilize its alignment by deepening the bed.

A point to be given due consideration is the location of the barrier relative to the mouth of the canyon. If débris storage is accomplished at points only a short distance above the mouth, the stream will reach the cone practically free of débris. In heavy floods, such a stream would have the capacity to pick up and move even the largest type of débris. The general effect would be a deepening of the stream bed in the upper part of the cone, a re-grading of the materials removed, and their transport to points beyond the former margin of the cone. It is conceivable that this last-named activity might be the cause of property damage.

On the other hand, if the barrier is placed at some distance above the mouth of the canyon, the scouring action of the de-loaded stream would be primarily on the intervening section of the river bed, with the effect on the cone less accentuated. This result may be expected in cases where the stream intercepts below the barrier a number of tributaries which furnish a substantial supply of débris.

Over a period of moderate floods, the changes in the stream bed below the débris storage may be minor and gradual, while during capital floods they may be expected to become very marked and may be permanent. The essential fact remains that débris storage in the canyon involves a radical change of natural processes and hence will have its lasting effects on the action of the stream on the cone.

Practical Application of Débris Storage.—The practicability of débris storage behind dams was recognized and adopted years ago by the California Débris Commission to prevent devastation of valley lands by the waste from hydraulic mining operations. Numerous débris dams have been constructed and are being operated successfully in the western foothills of the Sierra Nevada. Of late years, débris barriers have also been used as flood-control measures in Los Angeles County, California, and in Utah.[5]

In the report of the Special Flood Commission[6] appointed by Governor George H. Dern, of Utah, it is stated that the present situation in Davis County is such as to call for steps: (*a*) To erect control works at the mouths of the canyons that have flooded, commensurate with the property values involved; (*b*) to control the débris that will be washed on to the farm lands and other property at the mouths of the canyons by high water in the spring, and by summer freshets; and (*c*) in so far as possible, to control summer floods that may occur until such time as adequate vegetation conditions have been restored on the water-sheds.

The essential requisite for débris storage is an economic dam site, since a reservoir site such as that for economic water storage, is not the governing requirement. The slope which débris will assume behind a barrier can be estimated from a study of canyons with alluvial fills. In such canyons, the gradient of the stream is the combined result of stream flow and débris. These two factors are not affected by the construction of a dam or barrier, which accounts for the tendency of the stream to re-establish the gradient

[5] "Check Dams Control Débris Movements on Mountain Streams," by L. M. Winsor, M. Am. Soc. C. E., *Engineering News-Record*, August 20, 1931.

[6] "Torrential Floods in Northern Utah," *Bulletin*, Agricultural Experiment Station, Logan, Utah, 1930

it had before. As a matter of fact, the cone below the canyon mouth assumes the function of the barrier, above which the canyon fill has accumulated, and it is reasonable, therefore, to conclude that the slope ultimately produced above a barrier will not be materially different from that maintained by the old stream bed, except as modified by the greater width of the canyon. The effect of the latter would tend to produce a steeper slope because of less favorable channel properties.

In selecting a site for a barrier, this fact has a bearing on the ultimate débris capacity in that a location below a wide section of the canyon would increase the available space, not only laterally but also vertically, because the new deposit would assume a steeper gradient. Fig. 2 shows the effect of a dam 6 ft high (built in 1921 across one of the active flood channels) on the

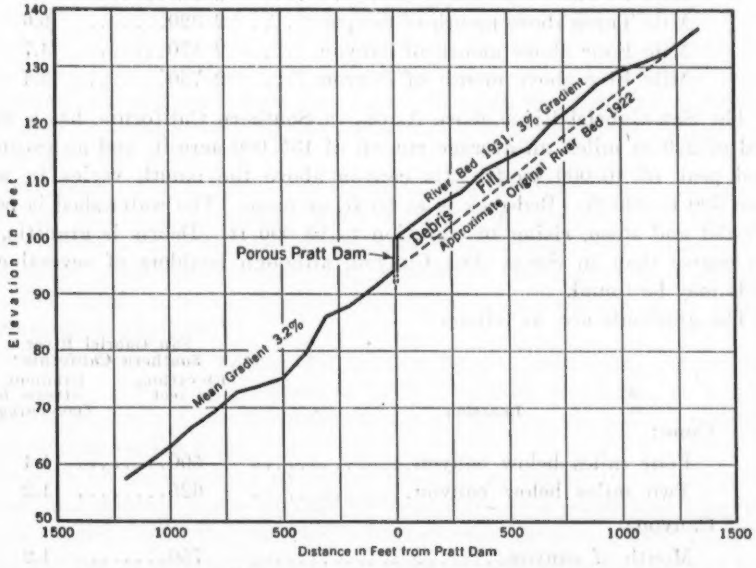

FIG. 2.—EFFECT OF INSTALLATION OF PRATT POROUS DAM ON CONE OF SANTA ANA RIVER, SOUTHERN CALIFORNIA.

cone of the Santa Ana River near San Bernardino, in Southern California, for the purpose of diverting water on to adjacent spreading grounds. The original gradient was approximately 3.2 per cent. The débris is exceptionally coarse, consisting mainly of large boulders. The survey of 1932 shows that the channel above the dam has been filled to a gradient of 3% for the first 1 000 ft.

The following examples will indicate what gradient may be expected. The Santa Ana River, above Mentone, in San Bernardino County, Southern California, has a well forested water-shed of 189 sq miles. Elevations range from 2 000 to 11 000 ft; and mean run-off is 75 000 acre-ft per annum, the estimated peak flood flow being 29 100 sec-ft, with a mean velocity of 13 ft. The width of the canyon varies for the first 4 miles from 500 to 700 ft. Bed-

rock is at 70 ft, more or less. Débris is granitic and exceedingly coarse, with large boulders predominating. The stream gradients are as follows:

Location	Santa Ana River, Southern California: Elevation, in feet	Gradient of stream bed (percentages)
Cone:		
Mile Two below canyon	1 550	3.8
Mile One below canyon	1 800	3.0
Canyon:		
Mouth of canyon	1 875	1.2
Mile One above mouth of canyon	1 960	2.2
Mile Two above mouth of canyon	2 100	3.8
Mile Three above mouth of canyon	2 320	5.6
Mile Four above mouth of canyon	2 570	4.7
Mile Five above mouth of canyon	2 750	3.4

The San Gabriel River above Azusa, in Southern California, has a water-shed of 220 sq miles, an average run-off of 135 000 acre-ft, and an estimated flood peak of 40 000 sec-ft. Its canyon above the mouth varies in width from 300 to 800 ft. Bed-rock is at 60 ft, or more. The water-shed is poorly forested and steep, rising in elevation to 10 000 ft. Débris is granitic, and less coarse than in Santa Ana Canyon, although boulders of several cubic yards may be found.

The gradients are, as follows:

Location	San Gabriel River, Southern California: Elevation, in feet	Gradient of stream bed (percentages)
Cone:		
Four miles below canyon	500	1.4
Two miles below canyon	625	1.2
Canyon:		
Mouth of canyon	750	1.2
Mile Two, up stream	875	1.6
Mile Four, up stream	1 050	0.9
Mile Six, up stream	1 150	1.4
Mile Eight, up stream	1 300	1.6

Depending on the character of the débris, gradients as steep as 4% may ultimately be realized. The steep slopes of streams running on bed-rock cannot be maintained with alluvial fills. Where alluvial fills alternate with bed-rock exposures, the latter are characterized, as a rule, by steeper slopes.

The manner in which débris will be deposited above a dam is governed by the general laws of stream flow. In describing the process, the California Débris Commission states[7] that when the first of the débris arrives at the head of the (water) reservoir, where the current is relatively low, it will tend to

[7] "Hydraulic Mining Investigations in California," Senate Doc. No. 90, 70th Cong., 1st Session, p. 20.

pile up and thus flatten the slope. This will cause a progressive deposit up stream until the entire bed and slope of the river up stream from the reservior are raised. Thereafter, according to the Commission, feeding into the reservoir will come from material carried along or scoured from this bed.

When the débris enters the reservoir, all the tractional load will settle immediately, assuming a steep natural slope down stream, with its top surface in prolongation of the surface of the débris bed in the river above. The suspended material will enter the reservoir with the initial velocity approaching that of the stream, and all but the lightest material will then begin to settle immediately, a portion on the slope of the tractional material and the remainder prolonging and flattening the toe of this slope on the bed of the reservoir to a distance depending on the fineness and specific gravity of the material. The reservoir will thus be filled from the upper end by an extension of the bed of the river into the reservoir, similar to the formation of a delta, with a relatively steep slope at its down-stream end. "The cross-section of the reservoir down stream from this slope," to quote the Commission directly, "will then be reduced only by the deposit of silt until the toe of the delta slope reaches the dam."

It is clear, without further explanation, that the final stable surface of the débris will be that assumed under maximum flood velocities. The foregoing discussion indicates that under suitable conditions, débris storage over a period of years may assume large proportions and that with a series of dams the stream may be stabilized on the cone proportionately as deposit areas are made available.

It is considered imperative that barriers be of a permanent type of dam construction. The sudden release of large quantities of alluvia, as a result of dam failure, might create conditions far worse than the natural meandering of streams on the cone.

As with all flood-control structures financed by public funds, the people assume, and possibly justly so, that the works are of a permanent type, that they do control floods, that they will afford effective protection (any intention of the planner or any publicity to the contrary notwithstanding), and that, therefore, the use of areas formerly menaced by floods may proceed with safety. This psychological effect is accentuated where structures are beyond the easy access by the general public, and it gains with time as minor floods are handled successfully. For this reason, temporary structures that are likely to fail during major floods may augment the dangers incident to the ravages of floods which occur only at long intervals. Débris barriers should be built to maximum heights consistent with funds, and should be provided with spillways and drain outlets. Economic dam sites are found in almost any canyon.

Rate of Débris Production.—Data on débris production in mountain water-sheds are meager. Table 2 was compiled from various sources.[8] Referring to Item No. 2, Column (7), the capacity of Castlewood Reservoir,

[8] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1732, and Vol. 95 (1931), p. 1061.

which was originally 5 267 acre-ft, is said to have been greatly reduced after thirty-eight years. Had the reservoir been filled in this period, the rate of filling would have been 0.83 acre-ft per sq mile, annually.[9] Of the many streams mentioned in Table 2, all except Buckhorn Creek and Cherry Creek drain alluvial plains. Information on these two streams obtained by courtesy of Ralph I. Meeker, M. Am. Soc. C. E., is as follows:

TABLE 2.—Silt Deposits in Reservoirs

Item No. (1)	Reservoir (2)	Stream (3)	State (4)	Drainage area, in square miles (5)	Record, in years (6)	Silt deposit, in acre-feet per acre per square mile (7)	Silt by volume, in percentage of annual run-off (8)
1	Buckhorn......	Buckhorn Creek..	Colorado...	130	18	0.24	....
2	Castlewood.....	Cherry Creek...	Colorado...	165	38	0.83	....
3	Elephant Butte..	Rio Grande......	New Mexico	30 000	11	0.665	1.66
4	Roosevelt.......	Salt............	Arizona....	5 760	20	0.875	....
5	White Rock......	....	Texas......	114	5	1.19	....
6	Zuni...........	Pecos...........	New Mexico	500	22	0.88	1.88
7	Lake Wichita...	Holliday Creek..	Texas......	135	25	Very little	....
8	Lake Penick....	Clark Fork, Brazos River	Texas......	2 250	7	0.06	....
9	McMillan......	Pecos River.....	New Mexico	22 000	10	0.06	0.8
10	Worth.........	Trinity River...	Texas......	1 870	13	0.57	....

Buckhorn (Item No. 1) is a channel reservoir on Buckhorn Creek, a tributary of the Big Thompson River. Its contributing drainage area extends from Elevation 10 500 to Elevation 5 300 at the water surface, but most of it is below Elevation 7 500. The annual precipitation is from 20.00 to 16.00 in. The foothills are grass-covered with some bushes, while the mountain area is covered with pines, spruces, and quaking aspen. Geologically, the foothills are characterized by sedimentaries, ranging from cretaceous to carboniferous material and the mountain area is composed of metamorphic rocks, granites, and predominant schists. The 10 miles of stream channel immediately above the Buckhorn Reservoir traverses sedimentaries that are the chief source of erosion. There are no measured records of flow, the average of 5-year estimates by the Water Commissioner ranging from 2 000 to 51 000 acre-ft per year. The yearly average run-off may be accepted at 19 000 acre-ft.

Castlewood (Item No. 2) is a channel reservoir on Cherry Creek, a tributary of South Platte River. Its contributing drainage area extends from Elevation 7 600 to Elevation 6 500 at the water surface. The annual precipitation is ± 18 in. The cover is mostly grass, with small areas of oak brush and some pines. The surface geology is characterized by sedimentaries in the tertiary group, with loam soil and some rock outcrops. There are no measured records of flow, but it has been estimated by Mr. Meeker at 5 000 to 10 000 acre-ft, with an average of 7 500 acre-ft. Some lands in the basin are dry-farmed.

As to McMillan Reservoir, the period available for the determination of the silt load may not be representative, because the reservoir has been in

[9] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1732.

operation more than 40 years; it has been enlarged twice and loss in capacity may not have been in direct relation to run-off at all times.

Water Storage in Débris.—The material deposited in reservoirs draining alluvial plains may consist largely of silt and clay with initial porosity that may run as high as 60 or 70 per cent. It is quite different from that produced in mountain water-sheds of crystalline formation and composed of sand, gravel, and boulders, the pore space of which may vary from 8% for materials in which large boulders are predominant to 35% for sand deposits. The following values were determined experimentally for the San Gabriel Wash, in San Gabriel Valley, Southern California,[10] mentioned previously:

Miles below mouth of canyon	Material	Percentage of voids
2	Boulders, gravel, and sand	12
7	Coarse gravel and sand	17
10	Medium gravel and sand	20
12	Fine gravel and sand	28

A reservoir intercepting a stream such as the San Gabriel River would also receive small quantities of silt that otherwise would be deposited in the plains below the cone. The percentage of fine materials might increase considerably if the reservoir should intercept the débris of burnt-over areas, which as a rule deliver large quantities of silt and ashes, depending on the severity of the rain storm.

In semi-arid regions, the water storage in the débris above the barrier may assume commercial value. The tendency of the water-table is to assume a slope approximately parallel to the surface of the débris fill and, for this reason, an appreciable increment of ground-water over and above free-water storage is available. Even with relatively high rates of percolation, a retarding effect is produced and eventually seasonal water storage takes place. Provision for controlled drainage of the débris mass, therefore, should be made at the time the works are planned.

It is interesting to note that, offsetting a diminished water storage in reservoirs resulting from débris deposition, water storage in accumulated débris behind barriers increases in proportion to the quantity of débris.

It is concluded that a feasible method of stabilizing the course of the stream on the débris cone is available in the construction of suitable débris storage above the cone. In planning the location of such storage, it should be borne in mind that the débris plain is the combined result of stream flow and available débris. The complete elimination of débris will reverse stream activity from alluvial to erosive. The effect may turn out to be more radical and far-reaching than expected because of the steep gradients generally encountered and the large volume of the periodic floods. A location at some distance up stream from the head of the cone, which will intercept a part of the débris produced by the water-shed without completely eliminating it, may be worth consideration.

[10] "San Gabriel Investigation," by Harold Conkling, M. Am. Soc. C. E., *Bulletin No. 7*, Dept. of Public Works, State of California.

6.—Conclusions

(1).—Physiographical Balance.—The great constructive processes of Nature expressed in the functions of the water-shed cover, the stream, the débris cone, and flood-plain, combine in maintaining a physiographical balance. Any attempt to modify, or reverse, these processes by flood control or conservation measures will re-act, in due time, in disturbing this balance. Although the effect may be slow, and although it may appear to a fast-moving world as imperceptible or negligible, the change of natural forces thus initiated by permanent improvements may be lasting and irresistible; hence, they present new problems to solve.

(2).—Physiographical Inter-Relation of Mountain Water-Shed and Flood-Plain.—In the development of flood control or conservation programs, the agency undertaking the control of streams should be in a position to consider improvements in the mountain water-shed as well as on the flood-plain and to treat the stream system of a drainage area as a unit, because interference with natural conditions at one point on the stream may have its reaction on the physiographical balance at points below and eventually create a new menace which, sooner or later, must be met.

(3).—Co-Operation of Governmental Agencies Necessary to Attainment of Maximum Yield of Water-Sheds.—The demand for additional water supplies is responsible for numerous plans advanced from many quarters for the improvement of the water-shed, with a view to increasing its water yield. Some of these plans are considered impracticable, while others entail a risk tending to unbalance débris production and removal. The United States Forest Service and affiliated State agencies, while not approving some of these projects, have initiated systematic studies of the water-shed, however, with a view to increasing run-off and decreasing fire hazard. It is also noted that engineers, investigators, and agriculturists have come to realize that the water-shed is their common problem and that co-operation will lead to a rational solution of water conservation in the water shed.

(4).—Storage Regulation May Produce Flood Menace.—The regulation by storage of a major stream, in time, may become the cause of new inundation problems at the point of junction of tributaries below the reservoir, because the regulated flow may lack the power to move the load which an uncontrolled tributary may carry to the point of junction. This problem may present itself particularly in cases where storage of the capital flood flow is proposed. The effect of the Elephant Butte Dam on flood conditions of the Lower Rio Grande is an example.

(5).—Effectiveness of Débris Barriers.—The checking of débris transportation by a débris barrier will change stream activity below until a new load and a modified stream bed and gradients balance the carrying capacity of the stream. This principle may be applied to the stabilization of stream courses on débris cones and on the flood-plain, in that alluvial stream action may be lessened, stopped, or changed to erosive action.

Single barriers built to considerable height (50 ft, or more) are far more effective in intercepting abnormal débris loads which may result from

denudation or other causes, than large numbers of check dams built to nominal heights of about 6 ft.

(6).—Temporary Structures to Check Débris Not Advisable.—The checking of débris by temporary structures in the mountain water-shed is adverse to best public policy:

(*a*) For the physical reason that an abnormal débris load caused by failure of structures may unbalance stream activity and cause a shifting of the course of the stream, for which the agency erecting the barrier would be responsible.

(*b*) For the psychological reason that the people assume flood-control structures to be permanent and effective for any magnitude of flood. They take for granted that the natural flood channel below such works may be encroached upon and improved without risk.

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PAPERS

MODEL OF CALDERWOOD ARCH DAM

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Synopsis

A model built in the Aluminum Research Laboratories in New Kensington, Pa., of the Calderwood Arch Dam on the Little Tennessee River, south of Knoxville, Tenn., is the subject of the paper. The writers present a discussion of the theoretical principles on which the model study is based, how these principles are incorporated in the model, and the results of a series of tests made on the model in conjunction with some of the tests made on the prototype.

Theoretical Considerations

The behavior of an arch dam is so complex that in order to make possible a model study it is essential to determine a simple relationship between the two. This relationship is included in a set of conditions that are usually designated as similarity conditions. The writers have referred elsewhere to: (*a*) The mathematical derivation of the similarity conditions and the specifications that the material for an arch dam model must meet in order to satisfy these conditions[3]; (*b*) the discussions of these similarity conditions[4]; and (*c*) the work done in developing a proper material.[5]

The relationship between the behavior of the prototype and its model may be given by the ratios of the deflections and the stresses. Introducing the ratio of the corresponding linear dimensions of the model and prototype as the scale of the model, *R*, the similarity conditions stated in Reference (*a*) show that the simplest possible relationship of deflections and stresses are: First, to have the ratio of deflections of the prototype and its model equal to

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[3] "Theory of Similarity and Models," by B. F. Groat, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 96 (1932), Discussion by A. V. Karpov, M. Am. Soc. C. E., pp. 308–325.

[4] "Building and Testing an Arch Dam Model," by A. V. Karpov and R. L Templin, Members, Am. Soc. C. E., *Civil Engineering*, January, 1932, p. 11.

[5] "The Flexing of Rubber Products" by E. G. Kimmich, Am. Soc. for Testing Materials, 1932, Symposium on Rubber; and "Elastic Behavior of Vulcanized Rubber," by Prof. Hencky, *Transactions*, Am. Soc. M. E., 1933, APM-55-8, Discussions by A. V. Karpov and R. L. Templin.

the square of the inverse scale of the model, $\frac{1}{R^2}$, multiplied by the ratio of the specific gravities of the materials of the prototype and its model, $\frac{\rho_p}{\rho_m}$, multiplied by the inverse ratio of the moduli of elasticity of the prototype and its model, $\frac{E_m}{E_p}$; and, second, to have the ratio of the stresses of the prototype and its model equal to the inverse scale of the model, $\frac{1}{R}$, multiplied by the ratio of the specific gravities of the materials of the prototype and its model, $\frac{\rho_p}{\rho_m}$.

In order to have such simple relationship, the following similarity conditions stated in Reference (*a*) must be satisfied:

1.—The materials of the prototype and model must follow Hooke's law, which means that the modulus of elasticity, E, is constant in the prototype and the model. (A better expression for that condition is: The materials of the prototype and its model for their respective ranges of stress must exhibit a straight-line relationship between stress and strain.)

2.—The ratio of the specific gravities of the material of the prototype and its loading fluid, $\frac{\rho_p}{\gamma_p}$, must be equal to the same ratio of the model, $\frac{\rho_m}{\gamma_m}$.

3.—Poisson's ratios of the prototype and model materials must be equal at corresponding stresses, $\mu_p = \mu_m$.

4.—The ratio of the moduli of elasticity of the model and prototype must be equal to the scale of the model, R, multiplied by the ratio of specific gravities of the loading liquids of the model and prototype, $\frac{\gamma_m}{\gamma_p}$.

5.—The ratio between the ultimate unit tensile stresses that can be carried by the horizontal joints of the model and prototype must be equal to the linear scale of the model, R, multiplied by the ratio of the specific gravities of the loading liquids of the model and prototype, $\frac{\gamma_m}{\gamma_p}$.

6.—The ratio of the width of the vertical construction joints in the model and prototype, $\frac{b_m}{b_p}$, must be equal to the square of the scale of the model, R^2, multiplied by the ratio of the specific gravities of the loading liquids, $\frac{\gamma_m}{\gamma_p}$, and by the inverse ratio of the moduli of elasticity, $\frac{E_p}{E_m}$, of the model and prototype.

7.—The action of the indefinitely large foundation rock may be represented by a block of elastic material, of some definite dimensions, which has a fairly low modulus of elasticity, so that by supporting it on a base made

of a material having a high modulus of elasticity, the conditions of an elastic foundation block supported on an unyielding non-elastic base can be represented.

8.—The ratio of the temperature difference of the model material, between the time of testing and building of the model, and of the temperature difference of the material of the prototype, between the time for which the investigations are made and the time of the building of the prototype, $\frac{T_m}{T_p}$, must be equal to the inverse ratio of the coefficients of thermal expansion of the materials of the model and the prototype, $\frac{c_p}{c_m}$, provided these coefficients are sufficiently uniform to be assumed as constant.

In applying these conditions to the correct design of a model, it is more convenient to divide them into the following classes: (I) Conditions Governing the Methods of Loading the Model; (II) Conditions Governing the Properties of the Model Material; and, (III) Conditions Governing the Method of Building the Model.

(I) Conditions Governing the Methods of Loading the Model

There are two possible ways to load the model. The first is to simulate exactly the loading conditions of the prototype and to load the model by action of gravity forces (weights) of the model material and the loading liquid. The second is to attempt to simulate the loading conditions of the prototype by loading the model with properly distributed forces. A number of methods may be devised for this purpose, such as loading the model by means of a number of weights or springs, or by placing it in a centrifuge and, by rotating it, produce the load by centrifugal forces.[6]

By the first method a comparatively simple relationship may be established between the behavior of the prototype and its model, depending on the fulfillment of Similarity Conditions 1 to 8, and only water or mercury can be considered. Since mercury is excluded on account of the specific gravity limitation, water is the only suitable liquid. Any attempt to establish exact similarity conditions for any model loaded in accordance with the second method will result in such a complicated relationship between the behavior of the prototype and its model that it probably could not be applied to the general study of the behavior of an arch dam.

(II) Conditions Governing the Properties of the Model Material

The established similarity conditions, stated in Reference (*a*), show that a simple relationship between the behavior of the prototype and its model is possible only when the ratios of the specific gravities of the loading liquid to the material in the prototype are the same as in the model. To realize the importance of this similarity condition, the principle of superposition

[6] "Use of Models for the Study of Mining Problems," by Philip B. Bucky, *Technical Publication 425*, Am. Inst. of Min. and Metallurgical Engrs.

should be applied properly to a model dam. This well-known principle may be stated as follows:

Any increase or decrease in the loading will cause a proportional increase or decrease of the stresses and deflections, provided the deflections are sufficiently small.

If applied properly to the model this principle would mean that if the loading is increased by a simultaneous increase of the specific gravities, both of the loading liquid and the material of the model, the deflection and stresses will change proportionally. Under such conditions the corresponding deflections and stresses of the prototype can be readily evaluated.

An improper application would be to assume that the specific gravities of the loading fluid and material of the model may be changed independently and that the stresses and deflections would still remain proportional. If the principle of superposition could be applied in such a manner, the ratio of specific gravity requirement would be of no importance, and it would be a simple matter to recompute the stresses and deflections of a model that is built with an improper ratio of specific gravities of loading liquid and material and to predict the stresses and deflections of the prototype. For several reasons, however, the inter-relation between the weights of the loading liquid and the dam material must be taken into consideration.

The first reason is that, by taking the strictly theoretical point of view, and considering the vertical stresses in a vertical element of an arch dam (that is, the so-called cantilever stresses), if the straight-line distribution of stress is assumed, the principle of superposition could be applied independently and an increase or decrease in the specific gravity of the loading liquid or the material would simply increase or decrease the stresses in the proper ratio. However, the assumption of straight-line distribution of stress does not represent the actual stress conditions. An illustration of this point may be of interest. For such illustration, it is preferable to consider a dam with a cross-section, as shown on Fig. 1, which differs somewhat from the conventional. The difference consists in introducing fillets at the sections where the faces of the dam meet the foundation by making the foundation surface a continuation of these faces. Such a shape removes the lines of discontinuity that occur in the conventional dam cross-sections at points where its faces meet the foundation, and makes it easier to visualize the stress conditions.

If an attempt is made in such a case to follow the vertical stresses along the line, *A-B,* at the foundation, they are definitely known at only two points: At *A,* the vertical stress is equal to the total water pressure, $w\ h$, and at *B* it equals zero. No methods are known by which the stresses can be evaluated between these two points, but it is clear that in order to retain static equilibrium, a distribution of vertical stress of the order illustrated on Fig. 1 is necessary, and this stress distribution may be assumed as giving a fairly correct picture without attempting to show its relative magnitude. If, now, the specific gravity of the material of the dam is reduced and that of the loading fluid remains the same, the stress shown along the line, *A-B,* will change and the maximum tension and compression will increase as

indicated in Fig. 1 by dotted lines. However, the stresses at A and B will remain unchanged. In other words, there will be no proportionality of any kind in change of stress and, consequently, of deflection.

In order to retain the proportionality between stresses, and, consequently, between deflections, of the prototype and its model, it is necessary, in case

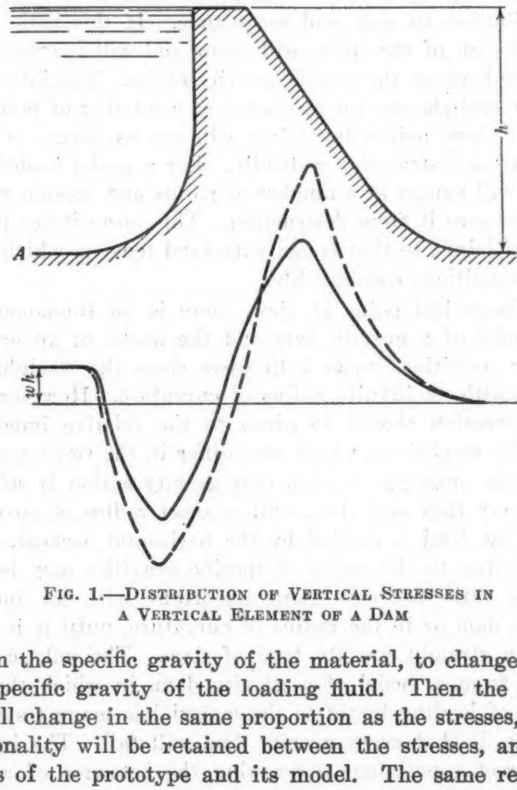

FIG. 1.—DISTRIBUTION OF VERTICAL STRESSES IN A VERTICAL ELEMENT OF A DAM

of a change in the specific gravity of the material, to change it in the same ratio as the specific gravity of the loading fluid. Then the pressure of the fluid, $w_f\ h$, will change in the same proportion as the stresses, and the necessary proportionality will be retained between the stresses, and also between the deflections of the prototype and its model. The same reasoning applies to the horizontal stresses at the points at which the horizontal arches meet the abutments or the bed-rock of the canyon.

The second reason for considering the relative weights of the loading liquid and the dam material is that the effect of the ratio between the specific gravities of the material of the model and the loading fluid is important when the resistance to overturning is considered for the dam as a whole. When subjected to water load an arch dam deforms elastically and tends to tip over as a unit. This tendency is opposed mostly by the resistance of the arch abutments. The ratio of the specific gravities will influence the proportion of load carried by the arches and transmitted to the abutments.

The third reason is a more practical consideration. Even assuming a straight-line distribution of stress, it is clear that if the ratios of specific gravities of the loading liquid and the material of the dam differ considerably

in the prototype and its model, a reversal of stress will occur at some points; that is, tension will appear in the model instead of compression at the corresponding point of the prototype, or *vice versa;* and, under actual conditions of stress distribution, similar to those in Fig. 1, more pronounced reversal of stress would occur. Only when the ratio of the specific gravities is the same in the prototype and in its model, will there be a complete similarity of stress distribution in sign and magnitude. If this ratio is different, a dissimilarity of sign of the stress will occur and will increase in proportion to the difference between the specific gravity ratios. The joints of the prototype practically exclude tension resistance at a number of points, but if tension appears at these points the joints will act as planes of discontinuity, changing the stress distribution radically. For a model loaded with a heavy liquid, tension will appear at a number of points and tension resistance must be introduced to save it from destruction. This necessitates the building of a model without joints, so that it can withstand tension, which again violates the similarity conditions considerably.

From the theoretical point of view, there is no fundamental difference between the model of a gravity dam and the model of an arch dam. The same similarity conditions cover both cases since the straight gravity dam is an arch dam with an infinite radius of curvature. However, in designing a model, consideration should be given to the relative importance of the various similarity conditions, which may differ in the two types of dams.

As long as the prototype is such that gravity action is subdued (as, for instance, in a very thin arch dam, with a short radius of curvature, and in which most of the load is carried by the horizontal arches), the similarity condition pertaining to the ratios of specific gravities may be of little importance. This ratio becomes more important with an increase in the thickness of the dam or in the radius of curvature, until it is of paramount importance in a straight gravity type of dam. The only conclusion that can be derived from a model of a gravity dam, in which the ratio of the specific gravity of loading liquid to the material is many times higher than in the prototype, is that every gravity dam will fail. The impossibility of arriving at correct conclusions concerning the behavior of a gravity dam based on such a model study applies also to thick arch dams.

Agreement between the calculated and measured stresses and deflections on models loaded with mercury has generally been considered sufficient proof that this particular similarity condition is of little or no importance. This statement is rather convincing, but analyzing it more carefully, it can be shown that it is probably incorrect since no attempt was made to compare directly the calculated deflections and stresses in an actual dam with the results of the model tests. Such calculations were made only for the model, which did not have vertical joints as in the prototype, and was loaded with mercury; consequently, the gravity action was considerably subdued and the arch action exaggerated.

Rubber-Litharge Compound Developed as Suitable Model Material.—The specifications covering a material suitable for the model may be derived

from Similarity Conditions 1, 2, 3, and 4. Using water as the loading liquid, the specific gravity of the model material is definitely fixed (in accordance with Similarity Condition 2) as 2.4; that is, the same specific gravity as the concrete in the prototype.

Similarity Condition 4 is of quite minor importance. It is derived from the requirement of exact geometrical similarity between the prototype and its model before and after the loads are applied. If the deflections of the model are not excessive, so that the loaded model does not change its geometrical shape appreciably, these conditions can be violated with the assurance that such violation will have little influence on the behavior of the model; furthermore, increased deflections facilitate model study.

Consequently, the following specifications for the material of the model were laid down in Reference (*a*)[7]: (1) Specific gravity of 2.4; (2) uniform elastic properties without pronounced directional qualities; (3) absence of skin or internal stresses; (4) low modulus of elasticity; (5) reasonably uniform values of the modulus of elasticity under varying unit stresses; (6) low permanent set; (7) low plastic properties; (8) properties that will permit the material to be provided readily in the required irregular shapes; (9) Poisson's ratio close to 0.20; and, (10) uniform coefficient of thermal expansion.

The rubber-litharge compound used in the model of Calderwood Dam meets all these specifications reasonably close except Specification (9), in which Poisson's ratio is 0.50.

The question then arises, "How will a model be influenced by this difference in Poisson's ratios?" If a gravity dam is located or constructed in such a way that the vertical joints permit enough lateral expansion to provide for lateral bulge of the material, the test results on a model built of such material will be close to actual conditions, the principal variation being the difference in deflections due to different shear moduli. If the conditions are such that they do not permit expansion of the material at the vertical joints, it is only necessary to determine the stresses in the model at a sufficient number of places, after which they can be expressed readily in terms of the prototype. In other words, for a gravity dam, the non-fulfillment of Similarity Condition 9 is of little importance. For an arch dam it may be of more importance, but the deviations in the behavior of the model will be comparatively small and can be accounted for to a large extent.

It was thought that this new material would permit the building of an arch dam model that would be not only a scalar representation of the prototype, but also an engineering model, the structural behavior of which is similar in every respect to the behavior of the prototype.

(III) Conditions Governing the Method of Building the Model

In the building of an engineering model of a dam, consideration should be given to the methods used in the building of the prototype and to the necessary adaptations of the model that should be made to insure the proper similarity of all conditions.

[7] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 322.

Similarity Conditions 5, 6, and 7 govern the building of the model. The exact fulfillment of these conditions is somewhat difficult, due to a number of uncertainties inherent in any arch dam, such as: Uncertainties of the dam proper (Similarity Conditions 5 and 6); and, uncertain boundary conditions of the dam where it meets the foundation and abutments (Similarity Condition 7).

Uncertainties of the Dam Proper.—Similarity Condition 5 establishes the ultimate unit tensile strength across the horizontal joints of the model and depends on the strength of the horizontal joints of the prototype. All existing arch dams are designed so that stresses will be considerably below the ultimate strength of the concrete. The horizontal construction joints, however, are usually subjected at a number of points to tensile stress that is higher than can be carried across such joints and, as a consequence, the joints open or crack, thus relieving such stress.

The tension that can be carried across a horizontal joint is rather uncertain and depends largely on the time interval between the placing of two successive layers of concrete. To ascertain the conditions at Calderwood Dam a few preliminary tests were made under conditions that simulated as closely as possible the actual conditions in the dam. These tests showed that the ultimate tensile strength of the horizontal construction joints that were tested, ranged from 60 to 88 lb per sq in. This justifies the conclusion that 25% of the ultimate tensile strength of concrete is about the maximum stress that can be carried in this manner and, in many cases, a much lower stress will open the joint. The horizontal joints in the prototype were spaced about 10 ft apart vertically and under such conditions no vertical tensile stress of any magnitude can be built up in the prototype. In order to simulate these conditions in the model, it was built so that no tensile stresses could be carried across the horizontal joints; but the number of joints was reduced, so that they correspond to a 20-ft vertical spacing in the model instead of the actual 10 ft in the prototype.

Similarity Condition 6 defines the width of the vertical construction joints of the model as compared with the prototype. It is certain that these joints in the prototype cannot carry any stress, but their width is somewhat uncertain.

In Calderwood Dam, as in every commercial dam, the concrete was not placed continuously as in monolithic construction, but in independent blocks, by placing in alternate piers. Considerable time elapsed from the beginning to the end of the concrete work. This procedure resulted in vertical construction joints, formed by the contraction of the concrete in the adjoining blocks. Customary methods were used to fill them by subsequent pressure grouting,[8] but due to the large size of the individual blocks, the time interval between the placing of the concrete and the application of the water load caused internal stress to be developed in the blocks of the prototype. Considering the large size of the blocks and the considerable tension that the concrete can withstand, it is probable that the internal tensile stress developed

[8] "Grouting Dam Foundations and Construction Joints," by James B. Hays, M. Am. Soc. C. E., *Civil Engineering*, November, 1933, p. 606.

in these blocks is rather high. Pressure grouting reduces the size of these joints, but probably has little, if any, effect on the internal stress developed in each block.

When the water load is applied, the concrete blocks are subjected to compression and then they contract. This contraction will be greater in a block that has a preponderance of internal tensile stress than in one without such stress. If the zero readings are taken before the first application of load, this additional contraction will be included in the measurements. The same conditions of increased contraction could be produced by building a dam of blocks without internal stress, but with wider vertical construction joints.

The rubber compound used in the blocks of the model dam had practically no internal stress when they were assembled. Any subsequent stress that has been introduced is due to differences in temperature of the model at the time of its construction and at the time of loading. Such stress, if any, is so small as to be negligible.

In order to retain the similarity conditions between the prototype, in which the concrete has internal tensile stress, and the model, in which the material has no such stress, the size of the vertical construction joints of the model must be made relatively larger than the theoretical size stipulated by the similarity conditions. The size of vertical construction joints is also influenced by the differences in Poisson's ratio for the material of the prototype and its model.

Uncertain Boundary Conditions of the Dam Where It Meets the Foundation and Abutments.—Similarity Condition 7 covers the condition at the boundaries; the deflections of the foundation and abutments of the model must be similar and in a proper ratio to the deflections of the prototype. The fulfillment of this condition would be fairly simple if sufficient information were available concerning the deflections of foundations and abutments in general and of the Calderwood site in particular. Little is known, however, about the deformation and stress in the bed-rock due to the dead load of a massive structure like a high dam and the additional stress and deflection superimposed by the subsequent water loading of the dam. The scarcity of information made it necessary to include a study of foundation deformation in the testing program.

In order to make such a study possible, the model was arranged so that the deflection of the foundation could be adjusted over a fairly wide range, and with as little disturbing influence on the model *per se* as possible. This increased flexibility of the foundation was obtained by applying screw-jacks, not under the model itself, but on the rubber compound foundation strip on which the model rested, the idea being that any unnatural change in stress or deflection resulting from such adjustments would be kept away from the model or at least smoothed out by the action of the flexible foundation strip.

The lower part of Fig. 2 shows the arrangement of the vertical and horizontal jacks which permitted increased flexibility in vertical and horizontal directions.

Comparative Prototype-Model Table.—Assuming the scale of the model as 1: 50 and basing it on the theoretical considerations discussed previously, as well as on the properties of the material of the prototype (see Appendix I)

FIG. 2.—HORIZONTAL JACKS TO CHANGE THE SIZE OF VERTICAL CONSTRUCTION JOINTS AND HORIZONTAL AND VERTICAL JACKS TO REGULATE THE FLEXIBILITY OF MODEL FOUNDATIONS.

and the properties of the material of the model (see Appendix II), the major factors are arranged for comparison in Table 1.

TABLE 1.—COMPARISON OF PROTOTYPE AND MODEL

Item No.	Description	Prototype	Model: Actual	Model: Required	Ratio, Prototype: Model
1	Deflections	....	....	....	1:2.68*
2	Stresses	....	....	....	50:1*
3	Strains	....	....	....	1:134*
4	Specific Gravities:				
	Loading liquids	1	1	1	1:1†
	Materials	2.375	2.380	2.375	1:1†
5	Modulus of elasticity, in pounds per square inch	3 770 000	563	563	....
6	Shear modulus, in pounds per square inch	1 640 000	188	245	....
7	Poisson's ratio	0.15	0.50	0.15	....
8	Coefficient of thermal expansion per degree Fahrenheit	0.000006	0.00008	....	....

* Theoretical. † Actual.

MEASUREMENTS MADE ON CALDERWOOD DAM

In connection with the study of the model of Calderwood Dam, four sets of measurements on the prototype are available, as follows: Measurements

made in April, 1930, before, during, and immediately after the filling period; and those made in September, 1930; May, 1931; and September, 1931. As the lake elevation has not been lowered materially since the initial filling in April, 1930, the dam has been continuously and quite uniformly loaded.

During each set of measurements the lake was maintained at approximately its maximum level, so that as regards the loading conditions, all four sets are similar. There is, however, a difference in temperature conditions.

The noticeable difference in the behavior of the dam as determined by these four sets of measurements can scarcely be attributed entirely to the temperature difference and indicates clearly that a number of additional factors influence the behavior of a dam. These factors may be referred to as "time effect," and the difference between the results of the subsequent tests probably represents the influence of such "time effect." The recognized factors that may produce "time effect" are: The plastic flow of the foundation, abutments, and the concrete of the dam; the difference in concrete temperatures caused by change in climatic conditions; the aging of the concrete; the volumetric changes in concrete caused by shrinkage due to aging; swelling due to the absorption of moisture from the lake or air; and shrinkage due to evaporation of water from the concrete. After a sufficiently long time a continuously and uniformly loaded dam should attain the final or permanent condition. After that, the behavior of the dam will depend only on the changes in climatic conditions.

The storage capacity of the lake behind Calderwood Dam is relatively small and it took less than six days to fill the lake and to load the dam fully.

Since the zero readings were taken immediately prior to the closure of the dam, when the water first began to rise, and the full load readings were taken immediately after the lake was full, the elapsed time between the dates of the two readings was short. The influence of the "time effect" can be noticed only after the dam has been loaded for some time. This combination justifies the assumption that, because of the short filling period, the "time effect" on the dam was so small that it did not influence its behavior. Such an assumption makes it possible to separate the "time effect" and study it in the light of subsequent measurements.

The present model study was made under the assumption of no "time effect" in conjunction with the measurements made on the prototype in April, 1930. The intention is to continue the model study in conjunction with subsequent measurements on the prototype, so as to obtain a better understanding of the influence of the "time effect."

During the first filling, three kinds of measurements were made on the prototype: Deflection, strain, and temperature measurements.

Deflection Measurements.—When the dam was built, provision was made for obtaining deflection measurements. A concrete pier was placed at the center of the crest circle and from this pier the distances were measured to a number of points on the down-stream face of the dam by means of a special apparatus and a measuring tape.

The apparatus consists essentially of the three-legged frame shown in Figs. 3 and 4, in which a swivel-frame is swung. The frame, in turn, supports a pulley over which the tape is reeved. The reference points for measurements are drilled on a block. Suspended by the tape, from the pulley, is a 120-lb weight hung from another pulley similar to that fastened to the

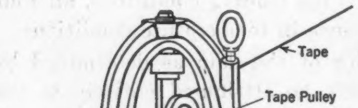

FIG. 3.—DETAILS OF DEFLECTION MEASURING APPARATUS.

FIG. 4.—OBSERVER MEASURING MOVEMENT OF TAPE

swivel arm. When raised from the floor of the pit in which it is suspended this weight subjects the tape to a constant tension of 60 lb. Ball-bearings are provided at either end of the frame as well as for both the tape and weight pulleys. The pulleys and the tape reel were made of aluminum alloys. The reel was constructed with internal reduction gears so as to reduce the force required to raise the weight, and was provided with a spring-brake to prevent the tape from leaving the reel or becoming loose and tangling. Means were provided to lock the reel so that it was not necessary to touch the apparatus while readings were being taken.

The measuring or reference block bolted to the swivel-frame had a shallow slot along one edge in which the tape moved. Four reference holes were drilled in this block, with a No. 57 drill (0.043 in. in diameter), one on either side of, but close to, the slot at the top and bottom of the block. The swivel arm was offset in such a manner that the edge of the tape reeved over the pulley, passed through the hole in the bottom of the swivel-frame and the center of the cross-member of the outer frame, lay in the slot of the measuring block, and coincided with the center of rotation. The other

end of the tape was provided with a hook that was inserted in stainless steel eye-bolts set into the down-stream face of the dam at each deflection measurement station.

Fig. 4 shows the offset of this swivel-frame and also serves to illustrate the method of measuring the movement of the tape, which corresponded to a movement or change in the deflection of the station on the down-stream face of the dam to which the tape was hooked. Each brass sleeve on which the foot graduations of the steel tape appear, was marked with suitable punch holes. The distance between the nearest of these marks and a corresponding reference hole on the measuring block of the apparatus was determined, using a standard vernier caliper, to each jaw of which was fastened a detachable hardened steel point, conical in shape. This change in distance could be determined to the nearest 0.001 in. Since the change in distance from the deflection station to the center pier was the desired measurement, the difference in reading of the vernier caliper, when adjusted to the same footmark and reference point each time, and properly corrected for temperature, angularity, etc., was a measure of the deflection of the dam at that point.

Making the deflection measurements at night and in fair weather it was possible to obtain the unusual over-all accuracy of $\pm$ 0.02 in. Since a single reference pier was used, the deflections could be measured in one direction only; that is, from the point on the down-stream face of the dam toward the reference pier. No provision was made to measure vertical displacements; hence it was necessary to assume that all deflection was horizontal, and to correct for the deviation. Due to the topographical conditions it was impractical to measure deflections at the lower elevations of the dam.

In this paper only the deflection measurements on the prototype made in April, 1930, are considered, the zero readings being taken after the dam was completed and before the water load was applied, and the load readings after the dam was loaded by the water in the full reservoir. That means that the measurements included in this paper show deflections due to water load only; or, in other words, the deflections caused by the dead weight of the dam are not considered.

Strain Measurements.—Strain measurements on the prototype were made on the down-stream and the up-stream faces of the dam. Those on the down-stream face were made at a number of rosettes where the strain was measured in four directions, vertically, horizontally, and in two diagonal 45° directions. These rosettes were set when the concrete was placed; each rosette consisted of eight steel inserts fastened to the inside of the forms on a gauge circle 10 in. in diameter. When the forms were removed, these inserts were anchored in the concrete. A small hole was drilled in each insert, and any change in distance between the gauge holes in two opposite inserts was measured with a 10-in. Whittemore strain-gauge.

The state of stress on the surface of a uniform solid may be obtained from strain measurements on three intersecting gauge lines, but at every rosette a fourth measurement was made which served as a check on the other three, and thus a number of inconsistent readings was eliminated.

Strain measurements on the up-stream, submerged face of the dam have not been made heretofore, although they are just as important—and may be even more important—than those on the down-stream face. Since there was no previous experience as a guide, an attempt was made to make such measurements at a few points.

The method consisted essentially of using a comparatively long gauge length (10 ft), at one end of which a piece of 0.091-in. invar wire was anchored to a heavy pin which was set about 2 ft into the concrete. (The invar wire had a modulus of elasticity of 20 300 000 lb per sq in.; a proportional limit of 14 000 lb per sq in.; a yield strength of 38 000 lb per sq in.; and an ultimate strength of 66 000 lb per sq in.) At the other end of the gauge length, this wire was attached to the short arm of a bell-crank lever anchored to the face of the dam by another pin which was machined to provide proper bearing. This bell-crank lever had a ratio of 5 to 1. To the long end of the lever was attached a piece of 0.048-in. galvanized hard steel wire which was carried almost vertically along the face of the dam and through a protecting pipe to the crest where the upper end of the wire was again attached to a lever and dead-weight arrangement so as to maintain a constant tension in the entire lever and wire system. (The hard steel wire had a modulus of elasticity of 29 000 000 lb per sq in.; a proportional limit of 140 000 lb per sq in.; and an ultimate strength of 180 000 lb per sq in.)

Corresponding to each vertical wire a second wire was fastened at its lower end to the lever fulcrum and at its upper end to a second lever-weight system. This second wire furnished a zero reference point in every case. The two wires leading up from any single gauge line were close enough to each other so that a microscope could be focused on first one and then the other, without moving the barrel of the microscope, except in the direction of the travel of the focusing screws. The use of the second wire automatically eliminated temperature corrections as well as corrections for movement of the top measuring point relative to the fulcrum of the lever. It was also possible with this set-up to measure the strains at all points with the same microscope by simply transferring the mounting. This mounting consisted of a vertical standard set into a block which could be clamped to the side of the guard-pipe by a hinged strap and a thumb-screw. A swinging arm fitted on the shouldered top of the vertical standard. The microscope was clamped directly to this swinging arm.

Fig 5 shows the arrangement of the pins and levers in the rosettes; Fig. 6 shows the wires entering the guard-pipe; and Fig. 7 shows the lever arrangement and microscope mounting at the top of the guard-pipe.

Strains were measured on three gauge lines at each station, one vertical, one horizontal, and one at 45° to the other two. A measurement consisted of determining the change in the difference in elevation between two scratches, one made on the stationary or dead vertical wire fastened to the fulcrum of the bell-crank, and the other made on the moving wire attached to the long arm of the bell-crank lever. Measurements were made with a precision filar microscope, which was mounted on the top of the guard-pipe in

FIG. 5.—ARRANGEMENT OF PINS AND LEVERS IN THE ROSETTES.

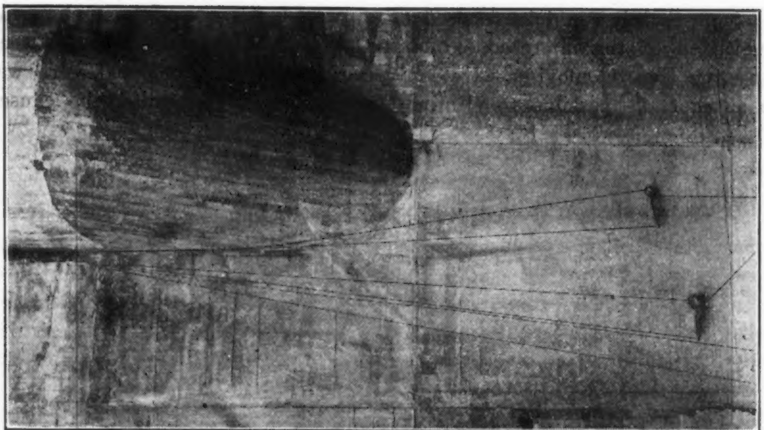

FIG. 6.—WIRES ENTERING GUARD-PIPE.

such a way that it could be focused on any pair of wires without changing the elevation of the microscope mounting. The interval of time between reading the live or moving wire and reading the dead or fixed wire was made as short as possible to eliminate any changes that might be caused

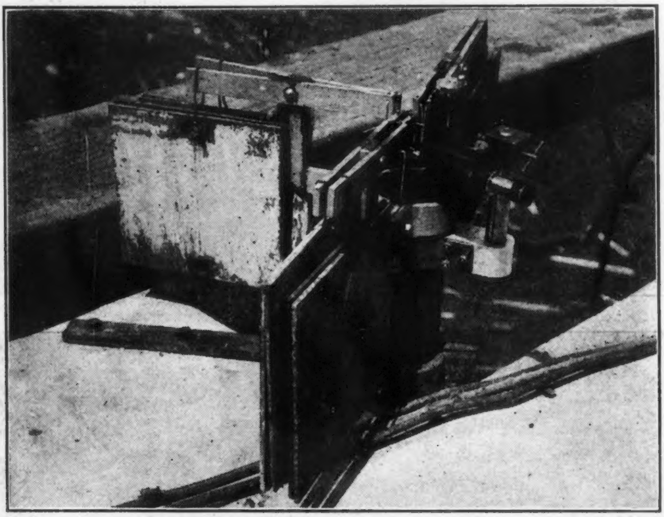

FIG. 7.—LEVER ARRANGEMENT AND MICROSCOPE MOUNTING AT THE TOP OF THE GUARD-PIPE.

from rising or falling temperature. One reading was made on each pair of wires of a rosette and repeated at least three times before moving to the next rosette. The results obtained showed conclusively that such measurements are possible and feasible if properly designed instruments are used.

In one case the strains were measured 160 ft below the surface. However, the limited number of these measurements made it impossible to form a correct picture of the stress distribution on the up-stream face of the dam.

As in the case of the deflection measurements, the strain measurements considered in this paper on both the down-stream and the up-stream faces of the dam were made so that the zero readings were taken after the dam was completed, but before the reservoir was filled. The load readings were taken immediately after the reservoir was filled. In so far as strains are concerned, the readings designated as "zero readings" do not represent the true no-strain conditions, but, on the contrary, the strain conditions, due to the weight of the concrete in the dam above the rosettes. In other words, the results of the strain measurements do not give the actual strains, but only the difference in strains which resulted from the application of the water load. Similarly, after the strains are reduced to stresses, they do not represent the actual stresses in the dam but only the difference due to the water load.

Temperature Measurements.—By means of a number of thermo-couples distributed through the prototype at different elevations and at different

distances from the faces of the dam, a rather complete picture was obtained of the temperature conditions. The temperatures were recorded automatically at definite time intervals during each set of measurements. The temperature differences of the concrete during the filling period, except at the surfaces, were small and, consequently, were neglected as were the other "time effects."

Measurements Made on the Model

In devising the method of making measurements on the model, it was kept in mind: (1) That such measurements could be simulated, reasonably well, on the prototype; and (2) that a number of additional measurements which could not be made on the prototype could be made on the model. Three kinds of measurements were made on the model, namely, deflection, strain, and temperature measurements, and these will be discussd separately.

Deflection Measurements.—Unlike the prototype, deflection measurements on the model could be made in any desired direction, so that true deflections could be evaluated. Fig. 8 and Fig. 9 afford a clear conception of the deflection measurements. Fig. 8(*a*) shows the section of the model and prototype at the elevation corresponding to Elevation 945.0 on the prototype, and gives the correct horizontal deflections of the model. The individual blocks are numbered and the vertical construction joints are indicated. The true deflections were determined by making measurements from two reference piers, one a permanent (center) pier and the other an auxiliary (removable) pier.

The deflection of a number of points on the down-stream face of the model was determined and deflection vectors similar to Fig. 8(*b*) were drawn to an exaggerated scale in the proper direction. The end points of these vectors were connected by straight lines, thus giving a diagrammatic representation of the deflection of the down-stream face of the model. Fig. 8(*a*) shows that in the middle horizontal section of the model, where the deflections are nearly radial, those measured toward the center pier approximate, closely, the true deflections. However, toward the abutments there is an appreciable difference between the radial and the true horizontal deflections.

Fig. 9 shows the condition at Block 10. The down-stream face of this block, indicated by the heavy full line, is drawn and the corresponding elevations of the prototype are indicated. As shown in Fig 9(*a*), the true deflection of a point at the down-stream face is determined by making two deflection measurements, one horizontal and the other at an angle to the horizontal. The deflection vectors of a number of points on the down-stream face of the model are drawn to an exaggerated scale in the proper direction and the end points of these vectors are connected by straight lines.

The dotted line in Fig. 9 gives the horizontal deflections on the model; the dashed line the true deflections which can be determined, provided the vertical deflections are considered. The solid line gives the deflections as measured on the prototype, multiplied by 2.68, the theoretical ratio of the model to the prototype deflections, to make them directly comparable with the model deflections. The prototype deflections are horizontal only. Here, again, it appears that near the top of the model the difference between the hori-

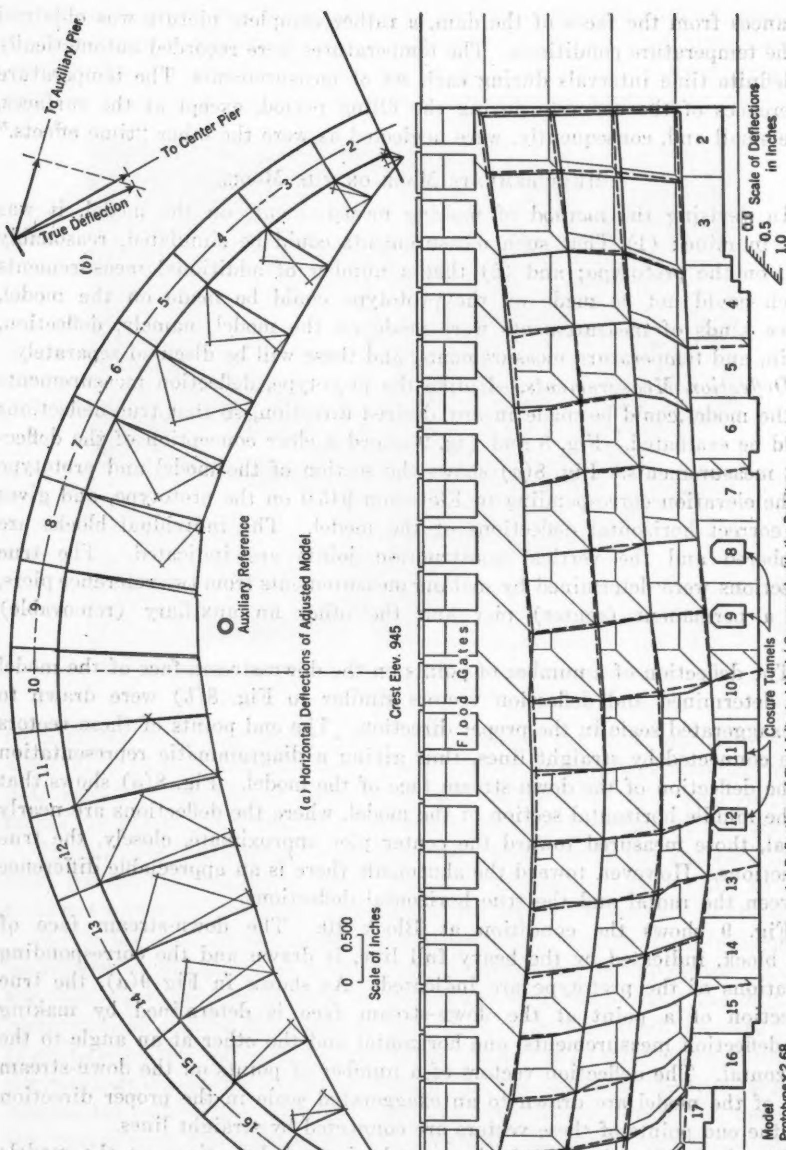

FIG. 8.—DEFLECTION MEASUREMENTS, CALDERWOOD DAM MODEL AND PROTOTYPE; (a) HORIZONTAL DEFLECTION OF ADJUSTED MODEL; (b) VECTOR DIAGRAM; AND (c) DEFLECTIONS OF FINAL ADJUSTED MODEL COMPARED WITH PROTOTYPE.

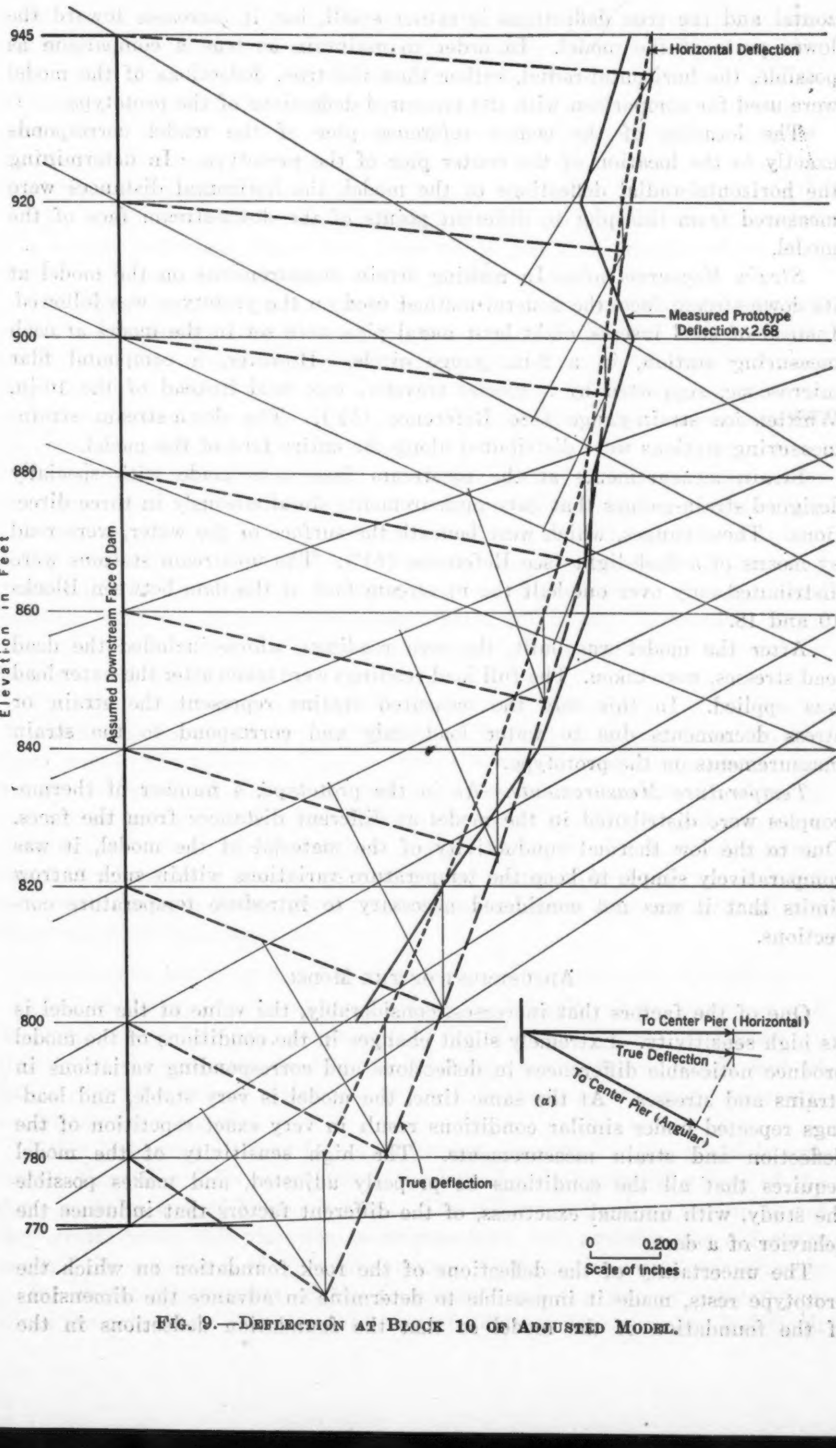

FIG. 9.—DEFLECTION AT BLOCK 10 OF ADJUSTED MODEL.

zontal and the true deflections is rather small, but it increases toward the lower parts of the model. In order to maintain as true a comparison as possible, the horizontal-radial, rather than the true, deflections of the model were used for comparison with the measured deflections of the prototype.

The location of the center reference pier of the model corresponds exactly to the location of the center pier of the prototype. In determining the horizontal-radial deflections of the model, the horizontal distances were measured from this pier to different points of the down-stream face of the model.

Strain Measurements.—In making strain measurements on the model at its down-stream face, the general method used on the prototype was followed. Instead of steel inserts, eight bent metal pins were set in the model at each measuring station, in a 2-in. gauge circle. However, a compound filar microscope, supported by a special traveler, was used instead of the 10-in. Whittemore strain-gauge (see Reference (*b*)[4]). The down-stream strain-measuring stations were distributed along the entire face of the model.

Strain measurements at the up-stream face were made with specially designed strain-gauges that gave measurements simultaneously in three directions. These gauges, which were beneath the surface of the water, were read by means of a flash-light (see Reference (*b*)[4]). The up-stream stations were distributed only over one-half the up-stream face of the dam between Blocks 10 and 18.

After the model was built, the zero readings, which included the dead load stresses, were taken. The full load readings were taken after the water load was applied. In this way the measured strains represent the strain or stress decrements due to water load only and correspond to the strain measurements on the prototype.

Temperature Measurements.—As in the prototype, a number of thermocouples were distributed in the model at different distances from the faces. Due to the low thermal conductivity of the material of the model, it was comparatively simple to keep the temperature variations within such narrow limits that it was not considered necessary to introduce temperature corrections.

Adjustment of the Model

One of the factors that increases, considerably, the value of the model is its high sensitivity. Extremely slight changes in the conditions of the model produce noticeable differences in deflections and corresponding variations in strains and stresses. At the same time, the model is very stable, and loadings repeated under similar conditions result in very exact repetition of the deflection and strain measurements. The high sensitivity of the model requires that all the conditions be properly adjusted, and makes possible the study, with unusual exactness, of the different factors that influence the behavior of a dam.

The uncertainty of the deflections of the rock foundation on which the prototype rests, made it impossible to determine in advance the dimensions of the foundation of the model so that the foundation deflections in the

model would correspond to those in the prototype. Consequently, it was necessary to devise some method of adjustment that would make it possible to vary the flexibility of the foundation of the model, so as to make its deflections agree with those of the prototype which contacts the foundation in two ways. In its central part the dam rests directly on the rock foundation and at the ends it contacts the abutments which, in turn, rest on the rock and re-act against it.

The model was built so that the arch proper, and its abutments, rested on a rubber-compound foundation strip of limited size, and the abutments, of the prototype were reproduced to scale in the model.

The deflections and stresses developed in the prototype may be considered as due to two factors: (1) Those that were developed after the dam was completed, but before the water load was applied (these are the dead load deflections and stresses); and (2), those that were developed by application of the water load (these are the water-load deflections and stresses and are the only ones considered in this paper).

The stress measurements on the prototype showed that the abutments of the dam received only a small part of the water load so that the resulting deflection of the foundation due to such load was limited. On the contrary, in the arch section the water load influenced considerably the deflections and stresses of the foundation. Consequently, in adjusting the model it was necessary only to introduce additional flexibility in the foundation which supported the arch section, without introducing additional flexibility in the foundation of the abutments.

The next adjustments pertained to the vertical construction joints, the size of which was increased by application of horizontal jacks shown on the upper part of the model in Fig. 2. These jacks produced the necessary upstream movement of the model and a corresponding increase in width of the vertical joints. Additional adjustments were made by introducing thin plate fillers in some of the vertical joints. The extent of such adjustment was determined by a number of preliminary tests. The final zero and full load readings were taken after these jacks had been removed and all the fillers had been introduced.

Comparison Between the Behavior of the Prototype and Its Model

If all similarity conditions are fulfilled, the behavior of the prototype and its model should be in exact agreement; but if some of these conditions are violated, a disagreement in relative behavior is introduced, which will be unimportant for minor violations, but will increase as the more important conditions are violated.

Table 1 shows that in so far as the rubber compound in the model is concerned, there is a difference between the actual and theoretical values of the shear modulus and of Poisson's ratio. The difference in shear moduli is not very great; hence this violation is unimportant; but probably the difference in the Poisson ratios is of more importance. There is also some uncertainty in so far as the adjustments of the foundation and the size of construction joints are concerned. This combination of different factors makes it difficult

TABLE 2.—STRAINS AND STRESSES IN THE PROTOTYPE AND ADJUSTED MODEL

Elevation		Block Number									
		17	16	14	12	10	9	8	6	4	2
(a) STRAINS, IN MILLIONTHS OF AN INCH PER INCH; DOWN-STREAM FACE OF PROTOTYPE (FILLING PERIOD, APRIL, 1930)											
940.0	Gauge Lines:										
	1			+10	+30	+10			+60	+24	+50
	2			—30	—30	—65			—20	—10	+20
	3			—50	—100	—135			—100	—23	—25
	4			—25	—40	—65			—40	+11	+15
920.0	Gauge Lines:										
	1					+15			+20		
	2					—40			—45		
	3					—75			—90		
	4					—10			—25		
900.0	Gauge Lines:										
	1		+13	+75	+83	+55			+45	+13	+20
	2		—57	+15	+28	+ 5			—35	+ 3	+60
	3		—18	—13	—28	—45			—110	—48	— 5
	4		+63	+63	+32	+20			—25	—38	—25
880.0	Gauge Lines:										
	1					+85					+15
	2					+22					+ 5
	3					— 2					0
	4					+70					+10
860.0	Gauge Lines:										
	1				+47	+87		+65	+45	—35	—10
	2				+ 2	+67		+40	— 5	—20	+25
	3				—30	+17		—10	—155	—65	— 5
	4				+40	+48		+15	—100	—80	—20
840.0	Gauge Lines:										
	1					+70					—30
	2					+55					—10
	3					+40					—15
	4					+40					—35
820.0	Gauge Lines:										
	1				— 3	—12			—15	— 8	
	2				—58	—52			—13	—13	
	3				—23	—80			—70	—28	
	4				+23	—40			—73	—23	
800.0	Gauge Lines:										
	1						—18				
	2						—13				
	3						—12				
	4						—17				
792.0	Gauge Lines:										
	1						—32				
	2						—19				
	3						— 7				
	4						—20				
(b) STRESSES,* IN POUNDS PER SQUARE INCH; DOWN-STREAM FACE OF PROTOTYPE (FILLING PERIOD, APRIL, 1930)											
940.0	Stresses:										
	V			+ 9	+58	—41			+173	+79	+178
	H			—187	—369	—516			—352	—76	—69
	S			98	213	238			266	85	125
920.0	Stresses:										
	V					+14			+26		
	H					—283			—337		
	S					157			184		
900.0	Stresses:										
	V		+39	+283	+304	+188			+109	+22	+75
	H		—62	— 5	—62	—141			—400	—177	— 9
	S		205	164	184	167			246	118	144
880.0	Stresses:										
	V					+327					+58
	H					+41					+ 9
	S					164					26
860.0	Stresses:										
	V				+164	+348		+245	+82	—173	—41
	H				—88	+117		0	—574	—271	—26
	S				139	118		131	358	112	75
840.0	Stresses:										
	V					+295					—124
	H					+196					—75
	S					56					49
820.0	Stresses:										
	V				—25	—93			—101	—48	
	H				—91	—316			—280	—114	
	S				138	115			131	36	
800.0	Stresses:										
	V						—75				
	H						—48				
	S						10				

* Compression is negative: tension is positive.

TABLE 2.—*Continued*

Elevation		Block Number 17	16	14	12	10	9	8	6	4	2
	(c) Strains, in Thousandths of an Inch per Inch: Down-Stream Face of Model										
940.0	Gauge Lines:										
	1	+4.8		+6.3	+7.5	+9.5			+6.7	+2.5	+4.9
	2	−4.6		−2.8	−2.3	−2.0			−3.8	+0.6	
	3	−8.5		−11.1	−11.5	−15.2			−11.4	−4.4	−10.7
	4	+0.3		−1.9	−2.4	−3.5			−1.9	−2.2	
920.0	Gauge Lines:										
	1					+5.9			+7.9		
	2					−1.8			−0.9		
	3					−9.1			−10.2		
	4					−1.0			−2.8		
900.0	Gauge Lines:										
	1		+3.9	+9.0	+8.0	+13.1			+5.8	+5.9	+2.8
	2		−3.7	−8.3	−1.4	+2.4			+2.5		
	3		−9.3	−11.8	−6.6	−10.9			−9.8	−7.2	−7.1
	4		0.0	+4.3	+2.7	−2.4			−6.9		
880.0	Gauge Lines:										
	1	+0.3				+6.3					+1.2
	2	−8.4				−0.5					+1.6
	3	−4.4				−8.9					−3.9
	4	+3.8				−2.0					−3.5
860.0	Gauge Lines:										
	1		+0.3	+1.9	+6.8	+8.3		+4.4	+6.3	+2.8	+4.9
	2		−3.7	−5.0	−1.3	0.0		+0.8	−1.5	+0.3	+1.6
	3		−3.9	−4.7	−9.1	−8.1		−7.3	−8.9	−4.1	−4.5
	4		+0.6	+2.6	−0.1	−2.0		−3.7	−3.0	−1.1	−1.7
840.0	Gauge Lines:										
	1	+5.2				+11.2					+3.5
	2	+10.2				0.0					+1.9
	3	−8.1				−10.2					−4.2
	4	−11.0				+1.2					−1.2
820.0	Gauge Lnes:										
	1			+3.0	+4.7	+7.5			+0.9	−0.5	
	2			−3.8	+1.6	+0.9			+0.8	−0.9	
	3			−5.6	−4.3	−6.3			−2.7	−0.9	
	4			+2.0	−2.2	+0.2			−3.2	−1.0	
800.0	Gauge Lines:										
	1					+4.9			−1.8		
	2					+2.8			+2.9		
	3					−1.8			−4.2		
	4					+0.5			−8.9		
	(d) Stresses,* in Pounds per Square Inch: Down-Stream Face of Model										
940.0	Stresses:										
	V	+0.41		+0.56	+1.31	+1.43			+0.75	+0.23	−0.34
	H	−4.58		−5.96	−5.81	−7.86			−6.04	−2.36	−6.19
	S	2.60		3.26	3.54	4.58			3.35	1.40	
920.0	Stresses:										
	V					+1.01			+2.10		
	H					−4.62			−4.69		
	S					2.79			3.35		
900.0	Stresses:										
	V		−0.56	+2.33	+3.53	+5.74			+0.68	+1.73	−0.56
	H		−5.51	−5.48	−1.95	−3.26			−5.18	−3.19	−4.28
	S		2.53	4.50	2.79	4.54			3.35		
880.0	Stresses:										
	V	−1.43				+1.39					−0.56
	H	−3.19				−4.31					−2.48
	S	2.42				2.83					1.34
860.0	Stresses:										
	V		−1.24	−0.34	+1.69	+3.19		+0.56	+1.39	+0.56	+0.99
	H		−2.81	−2.81	−4.28	−2.96		−3.83	−4.31	−2.02	−1.54
	S		1.12	1.86	2.94	3.07		2.32	2.94	1.30	1.86
840.0	Stresses:										
	V	+0.86				+4.58					+1.05
	H	−4.13				−3.45					−1.84
	S	4.65				3.98					1.53
820.0	Stresses:										
	V			+0.15	+1.91	+3.26			−0.34	−0.71	
	H			−3.08	−1.46	−1.91			−1.69	−0.86	
	S			1.94	1.83	2.57			0.97	0.08	
800.0	Stresses:										
	V					+3.00			−2.93		
	H					+0.49			−3.83		
	S					1.34			2.27		

* Compression is negative; tension is positive.

to set up any strict criteria as to the way in which the adjustment of the model should be made. It is a step-by-step procedure in which the influence of the different factors on the behavior of the model must be studied and the combination found which gives the most satisfactory results.

Deflections.—Fig. 8(*c*) shows the comparison of the deflections of the prototype and the adjusted model and represents the developed down-stream face of the model. The elevations correspond to those of the prototype. The block numbers are shown at the lower part of the dam. The deflections measured at the down-stream face of the model are shown by dotted lines. In order to make the prototype deflections directly comparable with those of the model, they were multiplied by 2.68, which is the theoretical ratio of model to prototype deflection. The deflections of the prototype are shown by solid lines.

Fig. 8(*c*) shows that, following the procedure just indicated, the deflections of the adjusted model are in a very satisfactory agreement with those of the prototype.

Strains and Stresses.—The data pertaining to the strains and stresses in the prototype and adjusted model are given in Tables 2 and 3. (In Table 2(*a*) and Table 2(*c*) Gauge Line 1 is vertical; Gauge Line 2 is at an angle of 45°; Gauge Line 3 is horizontal; and Gauge Line 4 is at an angle of 135 degrees. In Table 2(*b*) and Table 2(*d*), *V*, *H*, and *S* denote vertical and

TABLE 3.—STRAINS AND STRESSES, IN UP-STREAM FACE OF MODEL

Elevation	Block	Strains, in Thousandths of an Inch per Inch			Stresses†, in Pounds per Square Inch	
		Vertical	Diagonal*	Horizontal	Vertical	Horizontal
920.0	10	+7.8	—0.3	—14.5	+0.07	—8.34
847.0	10	+6.5	+0.3	— 8.0	+0.87	—4.61
787.0	10	+3.5	—1.3	— 1.5	+0.57	—1.31
916.0	12	+7.3	—5.4	— 2.8	+4.01	+0.26
876.0	13	+4.3	—9.5	— 9.5	—1.14	—6.31
919.0	14	+5.3	—1.3	— 5.3	+1.63	—2.41
807.0	14	+6.0	+0.3	0.0	+3.14	+0.89
845.0	16	+3.5	0.0	— 2.8	+0.55	—1.86
920.0	17	+1.5	—0.4	— 0.3	+0.67	0.00

* 135° from vertical. † Compression is positive and tension is negative.

horizontal stresses, and maximum shears, respectively.) Using these data, the strains and stresses in the adjusted model and the comparison between the prototype and its model may be shown by a number of diagrams, which represent the developed down-stream and up-stream faces of the model, on which are indicated the corresponding elevations of the prototype and the block numbers.

Fig. 10 shows both the direction and value of the principal strains on the down-stream face of the prototype and its model. With a few exceptions, the satisfactory agreement in the direction of the principal strains should be noted.

In order to make possible a direct comparison, all the measured data in Table 2 may be presented in a diagram in the terms of the model. Such diagrams will show the actual strains as measured on the model, but those

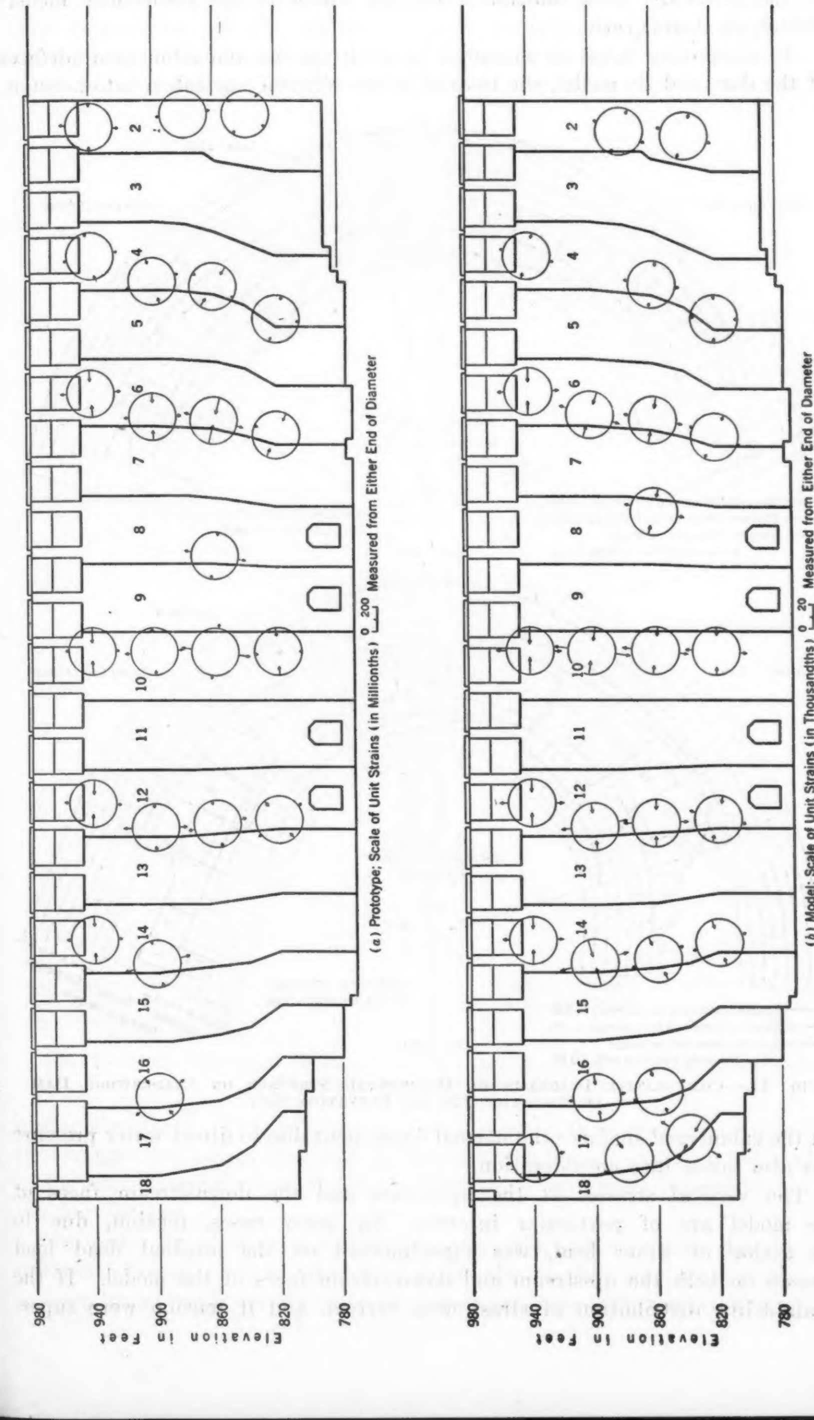

FIG. 10.—PRINCIPAL STRAINS AT THE FACES OF THE PROTOTYPE AND MODEL.

of the prototype were multiplied by 134 which is the theoretical model: Prototype strain, ratio.

In computing stress as measured by strain on the non-submerged surfaces of the dam and its model, the bi-axial stress relation was taken into account.

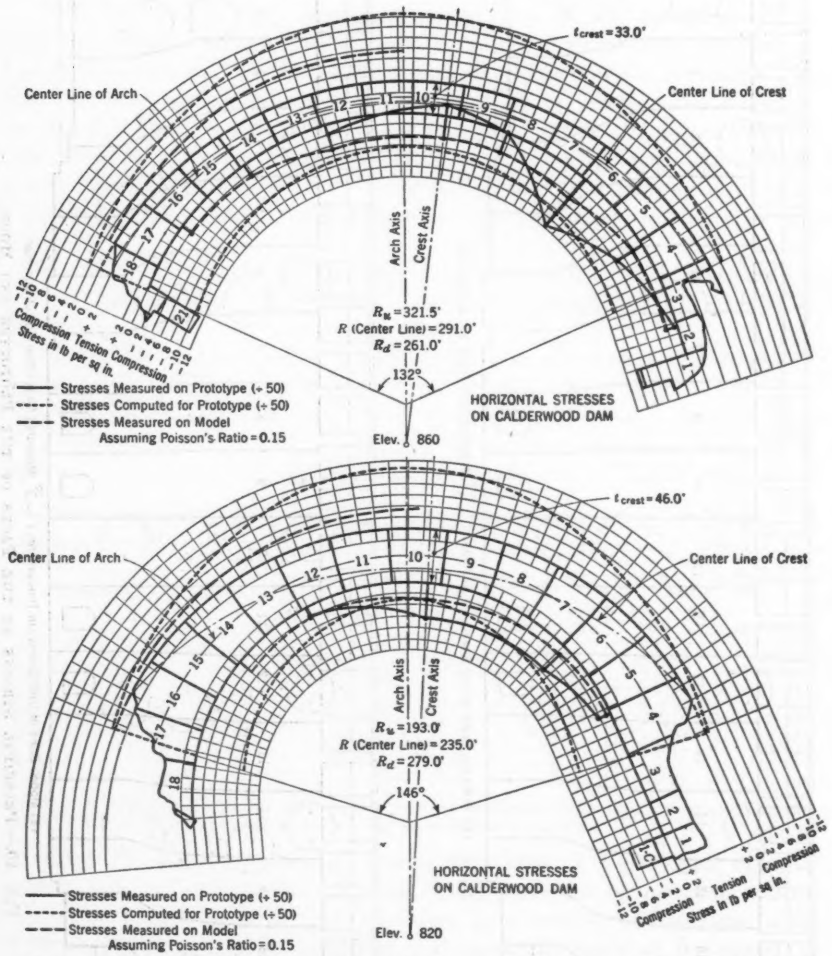

Fig. 11.—Comparative Diagrams of Horizontal Stresses on Calderwood Dam at Elevation 820 and Elevation 860.

On the submerged surfaces the normal component due to direct water pressure was also taken into consideration.

The vertical stresses at the up-stream and the down-stream faces of the model are of particular interest. In many cases, tension, due to the action of water load, was superimposed on the original dead load stresses on both the up-stream and down-stream faces of the model. If the straight-line distribution of stress were correct, and if tension were super-

imposed on one face, compression should be superimposed on the other face. The comparison of the vertical stresses actually superimposed on both faces of the model shows that the assumption of straight-line distribution of stress is incorrect, not only at the lower elevations, but probably also at the

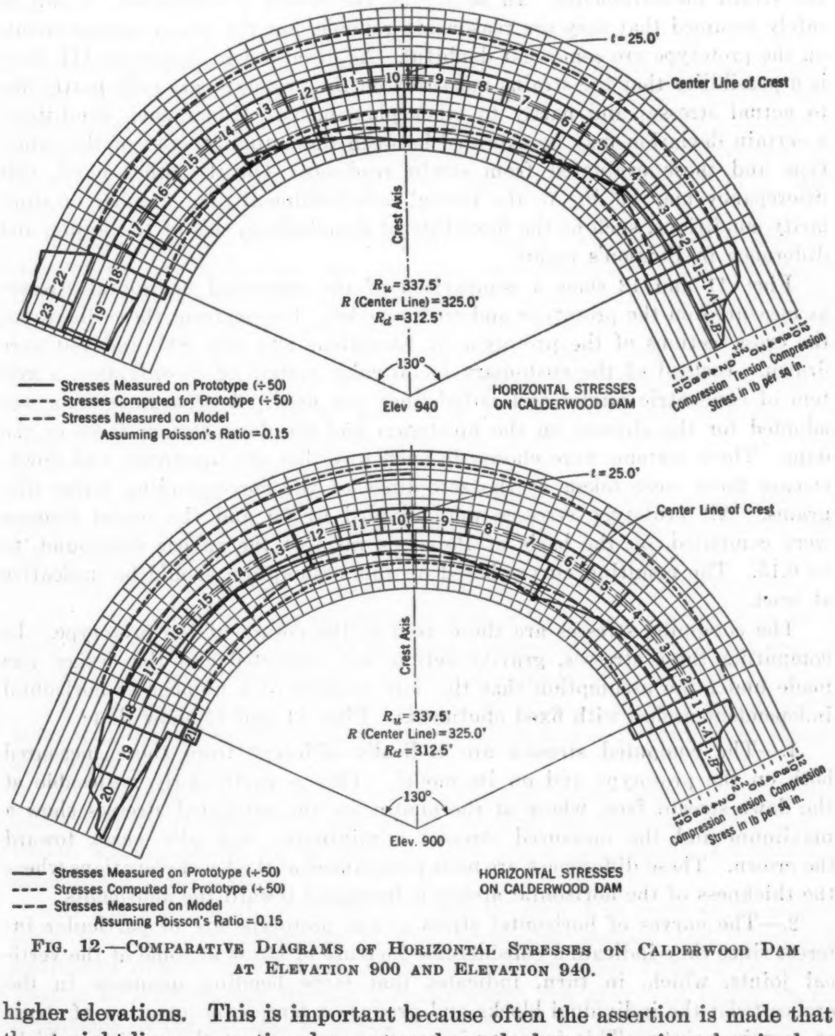

FIG. 12.—COMPARATIVE DIAGRAMS OF HORIZONTAL STRESSES ON CALDERWOOD DAM AT ELEVATION 900 AND ELEVATION 940.

higher elevations. This is important because often the assertion is made that the straight-line assumption does not apply at the lower elevations, but is close to the actual conditions at the higher elevations, which are influenced less by the foundations.

In order to eliminate, at least partly, the discrepancies due to the different Poisson ratios, the strains in the model may be reduced to stresses by taking Poisson's ratio for the model to be 0.15, the same as in the

prototype; the stresses of the prototype should be divided by 50 to have them correspond to the stresses in the model.

The agreement between the stresses is less satisfactory than in the case of the deflections. This discrepancy raises a question as to the reliability of the strain measurements. In so far as the model is concerned, it can be safely assumed that they are reasonably reliable, but the strain measurements on the prototype are somewhat doubtful. As discussed in Appendix III there is a possibility that the strains measured on the prototype are only partly due to actual stresses and partly to other phenomena. Under such conditions, a certain deviation may be expected between the actual stresses of the prototype and those evaluated from strain readings. On the other hand, this discrepancy may be due to the partial non-fulfillment of a number of similarity conditions, such as the flexibility of foundations, size of the joints, and difference in Poisson's ratios.

Figs. 11 and 12 show a comparison of the computed horizontal stresses as measured on the prototype and on its model. To construct these diagrams, the cross-sections of the prototype at Elevations 820, 860, 900, and 940 were drawn. Instead of the customary rectangular system of co-ordinates, a system of concentric circles and radial lines was used. A separate system was adopted for the stresses on the up-stream and the down-stream faces of the dam. These systems were chosen in such way that the up-stream and down-stream faces were taken as the zero lines of the corresponding stress diagrams. All prototype stresses were divided by 50 and the model stresses were computed, on the basis of Poisson's ratio of the rubber compound, to be 0.15. The resulting curves based on this assumption should be indicative at least.

The computed stresses are those used in the design of the prototype. In computing these stresses, gravity action was neglected and the design was made under the assumption that the dam consists of a number of horizontal independent arches with fixed abutments. Figs. 11 and 12 show that:

1.—The computed stresses are radically different from those measured both on the prototype and on its model. This is particularly noticeable at the down-stream face, where at the abutments the computed stresses show a maximum and the measured stresses a minimum, and *vice versa* toward the crown. These differences are most pronounced at the lower elevations where the thickness of the horizontal arches is increased toward the abutments.

2.—The curves of horizontal stress at the prototype are of particular interest since they indicate a considerable increase of stress at some of the vertical joints, which, in turn, indicates that large bending moments in the arches twist the individual blocks and produce a large concentration of stress at certain points. This increase is most pronounced at the sections which, due to the increased thickness, have horizontal arches that are very stiff at the abutments.

3.—In many cases the stress as measured on the prototype and on the model shows a fairly close numerical agreement and there is a similarity in the general shape of the stress curves.

4.—The non-uniform distribution of stress and in particular the reduction of stress toward the abutments on the prototype indicate that conditions might be improved by changing the shape of the horizontal arches.[9]

Conclusions

In order that a model study may be made to determine the behavior of a commercial arch dam, it is necessary to satisfy the major similarity conditions. The model described in this paper fulfills the most important ones and can be designated, with a considerable degree of accuracy, an engineering model. The pronounced similarity in behavior between the prototype and its model makes it possible to apply to the prototype the conclusions based on a study of the model.

The purpose of a proper design is to create a structure that is safe and economical. An arch dam, like any other structure, is safe and economical if the stresses are not only within the allowable maximum, but are uniform, and if, throughout the structure, they approach, closely, the allowable maximum stress. Too low stress at any point may indicate that the structure is not economical, and too high stress may indicate that it is unsafe. Finally, a non-uniform distribution of stress may make it necessary to reduce the average stress in order to keep the maximum stress low enough, and this may result in an uneconomical structure.

The tests on the prototype and its model show that there are a number of points where the stresses are too high and also too low, thus indicating that considerable improvement in the design of future structures can be made.

From the viewpoint of design, the following conclusions may be drawn:

(1) The foundation conditions have such influence on the behavior of an arch dam that a design which assumes a non-yielding foundation cannot satisfy the actual conditions and, therefore, is inadequate.

(2) The assumption of straight-line distribution of stress is not in agreement with actual conditions, and a design which is based on such assumption appears to be inadequate.

(3) The bending moments in the horizontal arches produce such heavy concentration of stress at some of the vertical construction joints that a design which neglects the influence of such joints and assumes a monolithic structure is inadequate.

(4) The circular shape of the conventional arch dam seems to be responsible for the non-uniformity of the horizontal stresses and introduces additional stresses which should be avoided. Presumably, this can be accomplished by substituting for the circular shape one that will better fit the conditions of the particular canyon.

(5) A design in which gravity action is neglected and all load is assumed to be taken by arch action results in a considerable and unwarranted increase in thickness of the horizontal arches at the abutments; the computed stress in such designs is radically different from the actual stress.

[9] "The Compensated Arch Dam," by A. V. Karpov, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1309; also, p. 1643.

(6) Vertical tension on both faces of the dam seems to be much larger than is usually assumed. The reduction of such stress merits attention.

From the viewpoint of building and testing a model, the following conclusions may be drawn:

(7) The adopted rubber compound permits the building of a model of any dam, reproducing its geometrical shape, the foundation contours, and the construction joints.

(8) Such a model will repeat its behavior under repeated loadings.

(9) The low modulus of elasticity of the rubber compound gives the model a high degree of sensitivity, and the relatively large distortions occurring under load permit the ready measurement of deflections and strains with a high degree of accuracy.

(10) The method of measuring deflections from three references (not in a plane) gives the absolute movements of any point at which deflections are measured.

(11) The method devised for measuring strains on the up-stream face, under water, and the arrangement for measuring strains on the down-stream face proved quite satisfactory as evidenced by the exactness with which readings could be repeated.

ACKNOWLEDGMENTS

The work described in the paper is a part of the arch dam investigation that was initiated by E. S. Fickes, M. Am. Soc. C. E., in an endeavor to put the design of such dams on a more scientific basis. The investigation is now being carried out by the writers, under the general direction of James W. Rickey and J. P. Growdon, Members, Am. Soc. C. E. H. N. Hill, Jun. Am. Soc. C. E., was directly responsible for the measurements made on both the prototype and the model. Recognition is also due to R. G. Sturm, Assoc. M. Am. Soc C. E., and to Messrs. C. Dumont and R. L. Moore, of the Research Staff of the Aluminum Company of America, for valuable assistance given in connection with the work.

APPENDIX I

PROPERTIES OF THE PROTOTYPE MATERIAL

In order to determine the proper relationship between the prototype and its model, it was necessary to secure reliable information concerning the behavior of the concrete of the prototype. The tests to determine the physical properties of this concrete were made on cylinders 2 ft. in diameter and 4 ft long, prepared as follows.

The concrete from which the specimens were made was taken from the batches that were being placed in the dam at the time of moulding the cylinders. It was dumped from the mixer into a 4-yd bucket, taken to the dam, and dumped on a flat-car, which was shifted to a side-track where the

concrete was shoveled into the moulds, first removing all stone larger than 6 in. The concrete was placed in three layers, each of which was vibrated for 1½ min. Immediately after moulding, the cylinders were covered with planks to prevent excess evaporation and checking. After 18 to 24 hours, water was allowed to run over the tops of all cylinders for 14 days, the forms being stripped within 1 to 3 days. The cylinders were then allowed to weather until ready for shipment to the National Bureau of Standards, United States Department of Commerce, Washington, D. C., where they were tested.

Modulus of Elasticity.—Stress-strain determinations were made on eight cylinders to determine the modulus of elasticity of the concrete. The average results were found to be, as follows:

Stress, in pounds per square inch	Average strain, in inch per inch	Stress, in pounds per square inch	Average strain, in inch per inch
0	0.0	600	0.0001615
100	0.0000279	800	0.0002125
200	0.0000542	1 000	0.0002665
400	0.0001063		

The stress-strain curve represents a practically straight line and the modulus of elasticity of the concrete of the prototype was found to be, $E_p = 3\,770\,000$ lb per sq in.

The exceptional regularity of this stress-strain curve seems to indicate that if the average values of the moduli of elasticity are considered, the material of the prototype satisfies perfectly the similarity requirement that the material must follow Hooke's law. At the same time it leaves the impression that for stress less than 1 000 lb per sq in., a properly mixed and vibrated concrete is a much more uniform product than is usually assumed.

Poisson's Ratio and Other Factors.—The average value of Poisson's ratio of the concrete was determined and was found to be 0.15. The shear modulus was not determined, but was computed in accordance with the formula,

$$S_p = \frac{E_p}{2\,(1 + \mu_p)} \qquad (1)$$

in which, E_p is the modulus of elasticity, and μ_p, Poisson's ratio of the concrete; and, from which, $S_p = 1\,640\,000$ lb per sq in.

The average specific gravity of the concrete was determined by weighing twenty-four test cylinders, and was found to be, $\rho_p = 2.375$. The coefficient of thermal expansion per degree Fahrenheit of the concrete was not determined, but was assumed as, $c_p = 0.000006$.

APPENDIX II

Properties of the Model Material

In securing the material for the model, it was clear from the tests that the laboratory specimens of the rubber-litharge compound satisfied the neces-

sary requirements. The laboratory testing methods have been described elsewhere.[10] The method of testing this compound was to cut three cylinders from the material in three different directions and apply the proper tests. This gave the necessary information regarding the uniformity of the material, but such methods are not well adapted to shop inspection and are practically unknown to the rubber industry. Therefore, it was necessary to develop a method, suitable for shop inspection, which would be familiar to the manufacturers; this method should be such as would insure a much higher uniformity of material than is customary in the commercial rubber compounds.

A number of preliminary tests indicated that for the inspection of a rubber compound manufactured under definite procedure and delivered in sheets of equal thickness, "durometer" readings give a satisfactory indication as to uniformity of the material.

Consequently, the uniformity criterion was established within narrow limits of variation in thickness, specific gravity, and durometer readings. These limits were, as follows:

Specific gravity.............................. 2.38 ± 0.03

Thickness of rubber slabs, in inches............ $1 \pm \frac{1}{32}$

In the total number of durometer readings:

Not less than 90% must be within ± 2% from the average reading

Not more than 10% may be within ± 3% from the average reading

Durometer readings were taken at each 4 by 4-in. square on both surfaces, and one reading at intervals of 4 in. along the edges of each slab.

After the material was received from the factory, a number of test cylinders were cut from slabs in three directions and the elastic properties were determined as described fully elsewhere.[10]

Modulus of Elasticity.—Stress-strain determinations were made on six sets of cylinders to determine the modulus of elasticity of the rubber compound. Each set consisted of three cylinders cut from the same slab. The average results of these determinations are, as follows:

Stress, in pounds per square inch	Average strain, in inch per inch	Stress, in pounds per square inch	Average strain, in inch per inch
0.0	0.00	10.25	0.01835
2.56	0.00461	12.81	0.02290
5.12	0.00921	15.37	0.02740
7.68	0.01380	17.92	0.03190

The stress-strain curve drawn from the foregoing data is practically a straight line, and the modulus of elasticity was found to be, $E_m = 563$ lb per sq in. Strain-stress curves of these data compared with those for the prototype show that in both cases the curves are straight lines within the limits of stress of the prototype and its model.

[10] "Methods for Determining the Physical Properties of Certain Rubber Compounds at Low Stresses," by R. L. Templin, M. Am. Soc. C. E., and R. G. Sturm, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. for Testing Materials, Vol. 31, 1931.

Poisson's Ratio and Other Factors.—The average value of Poisson's ratio of the model material was found to be 0.5. The shear modulus was not determined, but was computed and found to be $S_m = 188$ lb per sq in. The average specific gravity of the model material was determined by a number of tests, and was found to be $\rho_m = 2.38$.

For the material of the model the coefficient of thermal expansion per degree Fahrenheit was determined by a number of tests and found to be, $c_m = 0.00008$. Specimens of the model material were soaked in water from 10 to 14 days. The increase in weight and elongation due to this increase was negligibly small.

APPENDIX III

Suggestions Based on Experience Gained During the Work on the Prototype and Model

Considerable experience, gained during the progress of the work on the prototype and its model, was applied to the investigation, but some of it was gained too late to apply directly to this work *per se.* Nevertheless, this experience is of general interest, and the following suggestions may be helpful to those who expect to carry on similar investigation.

Measurements on the Prototype

First.—Deflection Measurements.—The method used to make the measurements from a central concrete pier by means of an invar or steel tape, gives satisfactory results, but this work must be done under favorable weather conditions, preferably at night. Measurement of deflection in the direction of only one pier does not seem to be sufficient; the use of two or perhaps three piers is recommended.

There is some indication that after the water load is applied, a certain movement of the entire foundation takes place in the vicinity of the dam, so that it is essential to tie in the reference points used for deflection measurements with other points at such a distance from the dam that they cannot be influenced by movement of the foundation.

Because of the foundation movement, deflection measurements made at the lowest elevations of the dam and, perhaps of the ledge adjoining the dam, should be of considerable value.

Second.—Strain Measurements.—The generally accepted methods of strain measurement seem to give results that are much less reliable than the deflection measurements. Even if it is possible to arrange the measurements in such a way that only a comparatively short time elapses between the zero and the full load readings, the changes that take place in the concrete may influence the strain readings considerably, so that the measured strains are due partly to stress and partly to the swelling and contraction of the

concrete. This is probably a particularly disturbing element during the first loading. If it were possible to load and unload the structure a number of times and take a number of zero and full load readings during the subsequent no-load and full-load conditions, this uncertainty could be removed, at least partly. Even then it is practically impossible to determine the actual stress in the concrete; probably only the stress change due to water load could be determined.

It seems necessary to devise a method by which it would be possible to determine, under any conditions, the load and zero strain readings at short time intervals. The best method would be to take a complete set of strain measurements at any desired point of the dam without the necessity of making any previous measurements under no load conditions. The method suggested for that purpose is illustrated in Fig. 13. The procedure would be as follows:

Drill at the proper places the necessary holes in the concrete of the dam and grout short steel plugs in these holes. The plugs are to be used as measuring points for the strain-gauge. Instead of drilling holes in the con-

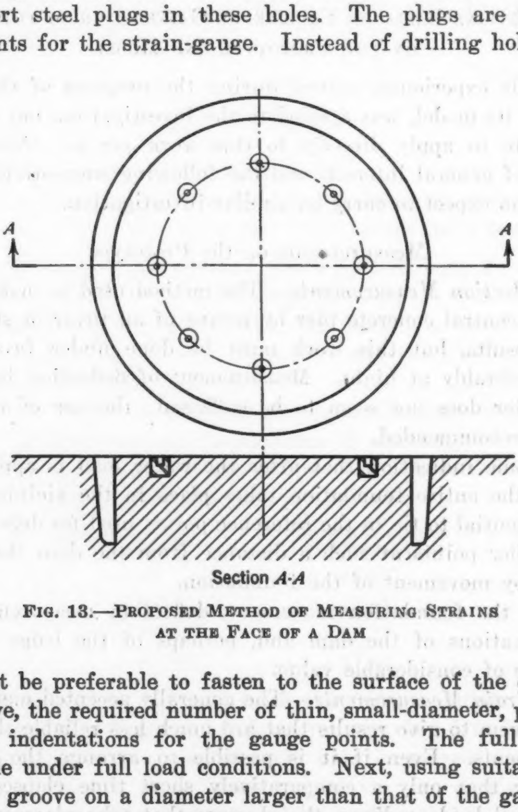

FIG. 13.—PROPOSED METHOD OF MEASURING STRAINS AT THE FACE OF A DAM

crete, it might be preferable to fasten to the surface of the concrete, using proper adhesive, the required number of thin, small-diameter, pieces of metal, with suitable indentations for the gauge points. The full load readings would be made under full load conditions. Next, using suitable equipment, cut a circular groove on a diameter larger than that of the gauge-line circle. The depth of the groove would be determined by check-strain readings. When a further increase of the depth of the groove does not influence such readings the desired condition of no surfaces stresses has been obtained.

The groove thus constructed will remove all stress from the surface of the concrete within the boundaries of the groove and, in this way, the zero readings may be obtained, not only under full water-load conditions, but without being influenced by the aging of the concrete, or by temperature changes, since the time interval between the full load and no load measurements may be short. Strain measured in this way will represent more closely strain due to stress alone. If the modulus of elasticity of the concrete is unknown a test cylinder may be cut from the dam at the same time and the modulus determined in the laboratory.

It is believed that the foregoing method would make it possible to determine the actual stress at any surface point not covered with water, on any existing dam, with great accuracy.

If it is deemed advisable to make strain measurements in the usual way, much longer gauge lines are suggested, and instead of the usual 10-in. gauge line, 5-ft, or even 10-ft, gauge lines are possible and preferable. Since such long distances cannot be measured by ordinary gauges it would be necessary to devise special instruments, which could be arranged so that the readings may be made at the top of the dam. It is of utmost importance to make strain measurements on both the up-stream, water-covered face of the dam and on the down-stream face.

Model Work

The rubber-litharge compound developed for the model of the Calderwood Dam seems to be superior to any other known material, but it still has the drawback of a high value of Poisson's ratio. Future progress in model work, which is so essential for the proper design of arch and gravity dams, depends to a large extent on developing a material similar in its properties to this compound, but with a much lower value of Poisson's ratio.

In designing a model the importance of a sufficiently extended foundation strip, particularly in the horizontal direction, should not be under-estimated. Improved methods of adjusting foundation deflections seem to be important. It does not seem necessary to reproduce a large part of the canyon, but rather to provide arrangements for adjustment which will make possible the simulation of a wide range of conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

WIND-BRACING IN STEEL BUILDINGS

THIRD PROGRESS REPORT OF SUB-COMMITTEE NO. 31, COMMITTEE ON STEEL OF THE STRUCTURAL DIVISION[1]

Introduction

Again, the Sub-Committee is indebted to those who in discussions have drawn attention to new aspects of the wind-bracing problem and who have thereby added to the knowledge of the subject. As a result of this helpful co-operative effort it now appears that a workable method of wind-bracing analysis and design, of greater accuracy than that which is at present attainable in estimating the applied wind force and in appraising the elastic action of composite structures, is within measurable distance of achievement.

In this Third Progress Report the matters considered are: (*A*) Comments on the discussion of the Second Progress Report[2]; (*B*) wind-bracing design; and (*C*) rigidity of riveted and welded wind connections.

(*A*) Comments on the Discussion of the Second Progress Report

(*1*).—*Prescribed Wind Force.*—It appears that there is still a considerable difference of opinion with respect to the wind force for which buildings should be designed. Mr. Robins Fleming[3] and L. E. Grinter[4], Assoc. M. Am. Soc. C. E., expressly approve of the Sub-Committee's tentative recommendation. Whatever the actual value of the prescribed force may be, H. V. Spurr, M. Am. Soc. C. E., would taper it uniformly down to zero at ground level from whatever value it has at one-third the height of the building[5].

L. J. Mensch[6], M. Am. Soc. C. E., thinks that "if the Sub-Committee found that buildings stood up under extremely higher pressures, it was its business to try to discover the hitherto undisclosed additional strength of buildings as a whole." Reference to the report of the Committee of the Structural Division on the Florida Hurricane of 1926[7] leads to the belief

Note.—Written discussion on this report will be transmitted directly to the Chairman of the Committee for possible use in preparing the Final Report.

[1] Presented at the meeting of the Structural Division, New York, N. Y., January 19, 1933.

[2] *Proceedings*, Am. Soc. C. E., February, 1932, p. 213.

[3] *Loc. cit.*, May, 1932, p. 947.

[4] *Loc. cit.*, p. 954.

[5] *Loc. cit.*, August, 1932, p. 1126.

[6] *Loc. cit.*, May, 1932, p. 944.

[7] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1118.

that the capacity of a building to resist a wind force much in excess of the designed wind load is vitally dependent on the strength and rigidity of the details. Where high flexibility exists, walls are likely to be shattered suddenly, with the effect of a suddenly applied load on the frame. If, however, the load is either entirely absorbed by the frame from the start, or is gradually transferred to it, the capacity of the structure to stand over-load is much enhanced. The Daily News Building, at Miami, Fla., in which care appears to have been taken to proportion both main members and details for rigidity as well as strength, was thus able to resist, with the aid of the walls and partitions, a wind force probably amounting to 50 lb per sq ft over moderate areas. Had the frame not co-operated effectively with the walls from the beginning, the latter would probably have failed suddenly with a shock such as sometimes accompanies the sudden shattering of a block of masonry under test. Consequently, the effect on the frame would have been the same as if a much higher wind force had been applied.

In questioning the validity of the grounds set forth in the second progress report in support of the prescribed wind force, Albert Smith, M. Am. Soc. C. E., mis-states the Sub-Committee's position somewhat. Rather than inviting the Society "to go at least part way with the New York Code Committee in a radical revision downward of wind loads,"[8] it is suggested that the Society recommend an increase in the wind force now specified, not only by the New York Code, but perhaps by a majority of the building codes of the United States. The Sub-Committee recommended as large an increase in this wind prescription (which, incidentally, has been approved by a great number of responsible engineers and building departments) as it felt justified in doing unless, or until, more positive evidence of the need for further increase becomes available than was at hand when the first report was formulated. While the primary concern of the Sub-Committee is structural safety, that safety should not be secured by setting up an extravagant demand with respect to wind, entailing unnecessary expense to the owners of buildings and increasing the difficulties involved in structural layout and design.

In relating the Sub-Committee's recommendation to the wind forces formerly required by building codes, Mr. Smith neglects to point out that the Sub-Committee ignores the effect of walls and partitions in strength calculations. Under the old codes, or in accordance with the official interpretations of them, as much as one-third to two-thirds of the prescribed wind force was considered as being absorbed by the masonry. Consequently, so far as the design of the frame and its details was concerned, the wind force assumed was often only 10 to 20 lb per sq ft, regardless of height, and without consideration of proper allocation of wind to braced bents, or of rigidity afforded by either main members or details. Moreover, the permissible stress allowed for members subjected to wind stress was often as much as 50% in excess of the basic working stress. As a matter of fact, therefore, the Sub-Committee is suggesting a stiffening up of current practice with respect to wind-bracing and not, as Mr. Smith represents, a letting down.

[8] *Proceedings*, Am. Soc. C. E., May, 1932, p. 950.

Doubt is expressed by Mr. Smith as to the relief from turbulence that exists in the case of a few tall towers rising to a considerable height above the general level of the adjoining buildings. Shelter does not arise wholly from blanketing. There may be upward or downward deflection of the air stream due to the highly irregular surface over which the wind passes. Interference with, and modification of, the gradient wind, therefore, must start at a considerable distance above the tops of the tallest structures present and must rapidly become more marked as the earth is approached.

S. P. Wing, M. Am. Soc. C. E., has made a valuable interpretation of the wind velocities observed at certain stations.[9] It is recognized, of course, that certain localities are more likely to experience higher velocities than others, but it is difficult to make a wind-pressure delimitation of the continent that would be generally acceptable. The suggestion that increased wind loadings should be prescribed for the New York area has already been met in effect, in that the Sub-Committee's recommended wind force is substantially more severe than that now required by the New York Building Code. Recognition of the possible need for making allowance for the shape of buildings and for suction effect was contained in Recommendation (3) of the first progress report.[10]

Like Mr. Wing, the Sub-Committee believes that structural security should not be allowed to depend on a secondary specification controlling rigidity. The problem is to decide whether the wind force tentatively recommended in the first progress report, having regard to all the circumstances and all the conditions imposed upon the design by the first and succeeding reports, will guarantee that security. It is the present belief of the Sub-Committee that, in applying to buildings in urban areas the results of observations taken in highly exposed locations, Mr. Wing has not made sufficient allowance for shielding and turbulence. Moreover, the magnitude of the prescribed force cannot be determined without recognition of all the collateral stipulations outlined in the second progress report.[11] Mr. Wing has ignored these when, referring to buildings in Miami, he observes that "some designed to the Sub-Committee's specifications survived and others failed." It is doubtful whether any buildings in the Miami area even approximately conformed to the Sub-Committee's recommendations, taken in their entirety.

(*2*).—*Permissible Stress for Combination of Wind with Other Loads.*— Jacob Feld[12], Assoc. M. Am. Soc. C. E., and Messrs. Fleming, Wing, and P. L. Pratley,[13] doubt the logic of specifying a lower permissible stress for members subjected to wind action alone than for those carrying a combination of wind with other kinds of loading.

Having regard to the relatively small probability of full specified live load occurring in members such as girders, for which the tributary area is large, the small probability of a full specified wind load on any large area in an

[9] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1103.
[10] *Civil Engineering*, March, 1931, p. 483.
[11] *Proceedings*, Am. Soc. C. E., February, 1932, p. 215.
[12] *Loc. cit.*, September, 1932, p. 1295.
[13] *Loc. cit.*, May, 1932, p. 958.

urban location, and the still smaller probability of the combination of maximum live load and wind load, it appears justifiable to permit an increase of permissible stress for the combination. The probability of the occurrence of maximum wind load being much greater than the probability of the concurrence of maximum live load plus wind load, the working stress for members carrying wind load only should properly be more conservative than that for the combination.

As pointed out in the second progress report, the object sought in recommending relatively lower permissible stresses for combinations in which wind plays an important part was to ensure that with any probable chance increase in wind load above that specified, the factor of safety would be measurably the same for members carrying a large percentage of wind as for those carrying a small percentage.

In its second progress report the Sub-Committee recommended that, in general, where the wind stress exceeds $33\frac{1}{3}\%$ of the sum of the other stresses (or 25% of the total stress) the permissible stress be progressively reduced as the proportion of wind increases, until for a member subjected to wind stress only the permissible stress is the same as that allowed for dead load, or for dead load and live load. The manner of effecting this reduction was left to the individual designer. Professor Grinter suggests the simple device of neglecting wind stress up to $33\frac{1}{3}\%$ of the other stresses and proportioning for the excess wind plus dead load and live load at the basic working stress for dead load and live load only. This method, which has been used for some years in certain specifications, is a satisfactory one, incidentally giving very nearly the same results as a linear reduction of working stress.

The manner in which the factor of safety is affected in case the design wind load is exceeded in service, is indicated in Fig. 1. Two cases are considered: (1) When the design for combined loading has been made according to the method prescribed in the specifications of the American Institute of Steel Construction; and (2) when it has been made according to the method recommended by the Sub-Committee.

Let the relative design factor of safety for dead load plus live load be unity. Then, when the wind load stress is 25% of the total stress (including wind) the relative factor of safety will be 0.75 according to either method of design. As the percentage that the wind bears to the total load increases, the relative design factor of safety, which has been designated as R, remains constant at 0.75 for Method (1), whereas under Method (2) it rises in linear ratio to 1.0 for the case where the member is subjected to wind stress only.

If, for a member in which one-fourth of the design load stress is due to wind, the wind load is exceeded by 50% in service, the member will then have $33\frac{1}{3}\%$ of its stress in the form of wind, and the relative factor of safety, R, under either method of design will be reduced to 0.67. Had the design load on the member been 100% wind, the 50% increase would have lowered R to 0.50 if the design had been according to Method (1), but would have reduced it only to 0.67 if the design had been according to Method (2). For members with any other percentage of the design load in the form of wind,

R, for Method (1), would have dropped to a value represented by the ordinate to the straight line the end ordinates of which are 0.67 and 0.50, while it would have remained constant at 0.67 for Method (2).

If it is assumed that the design wind load is exceeded by 100% in service, the reduced relative factor of safety, *R*, would range uniformly between 0.60 and 0.375 for Method (1) and from 0.60 to 0.50 for Method (2).

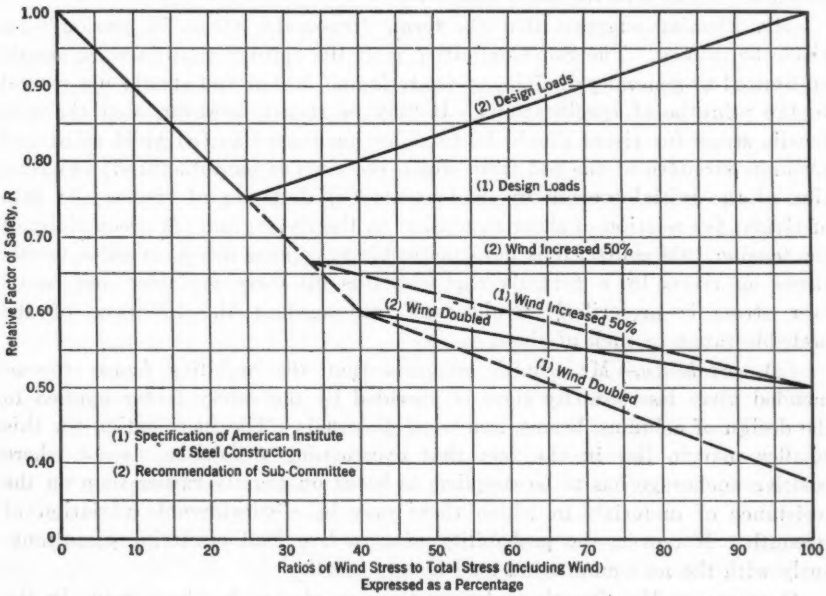

FIG. 1.—RELATIVE FACTORS OF SAFETY WHEN DESIGN WIND LOAD IS EXCEEDED.

From a consideration of these relations it is evident that some such design principle as that recommended by the Sub-Committee is necessary if, in the event of an increase in the wind load above its design intensity, approximately the same degree of security is to be maintained in members designed for different percentages of wind load.

(3).—Permissible Stress for Rivets and Bolts.—O. G. Julian,[14] M. Am. Soc. C. E., and Messrs. Fleming, Pratley, and Wing, question the wisdom of excepting rivets and bolts when recommending that members subjected to wind stress only be proportioned for the basic permissible stress.

The difference in treatment arises from the fact that there is a greater real factor of safety for rivets and bolts designed with due regard to the combination of wind and gravity loads and to the known behavior of joints under test than there is for members, particularly those under compressive stress. While on the basis of ultimate strength, at commonly prescribed basic working stresses, there is a factor of safety of about 2½ for compression members and 3½ for tension members, there is one of about 4 for rivets and

[14] *Proceedings*, Am. Soc. C. E., September, 1932, p. 1294.

bolts. Moreover, there is much less danger of sudden failure due to high stresses in rivets and bolts than in compression members and compression flanges which may fail suddenly by buckling or crippling. A secondary justification for the proposal is that the contribution of rivets and bolts to deflection is comparatively small, and, consequently, the use of higher stresses in them is less disadvantageous to the rigidity of the structure than the use of higher stresses in the main members.

Mr. Fleming suggests that the term, "reasonable stress in tension," for rivets be defined. The Sub-Committee is of the opinion that its work should be limited to general principles of analysis and design and should not extend to the minutiæ of specifications. It may be stated, however, that the safe tensile stress for rivets should be fixed having regard to (*a*) yield point and ultimate strength of the rod from which the rivet is manufactured; (*b*) relation of the initial tension to yield point; (*c*) diameter of rivets; (*d*) grip of rivets; (*e*) relation of shear to tension on the rivets; and (*f*) eccentricity of the tension. Of course, it is not practicable to express the permissible tensile stress on rivets by a formula that contains all these variables, but whatever stress is prescribed should take into account the influence of the probable range in each of them.

(*4*).—*Stability.*—Mr. Smith suggests that the stability factor recommended gives less security than is provided by the safety factor applied to the design of columns, beams, and wind diagonals. The justification for this smaller margin lies in the fact that overturning resistance, except where positive anchorage has to be supplied, is based on gravity rather than on the resistance of materials in which there may be a considerable variation of strength. Moreover, the probability of zero live load occurring simultaneously with the maximum wind load is very small.

Contrary to Mr. Spurr's understanding, no change has been made in the basic stability factor, which was retained at 1.5 in the second progress report. That report merely added to the first report the amplification that the factor be measured by the relation of the maximum wind-load tension to the dead load compression in the critical column. A further useful addition, suggested by Mr. Pratley, might be that the foundation itself must be made adequate for any uplift that may be imposed on it.

(*5*).—*Walls, Partitions, and Floors.*—It is very probable, as Mr. Mensch suggests, that under small wind loads the percentage relief in deflection and amplitude of vibrations afforded by walls and partitions is less than it is under large wind loads. The fact remains, however, that under no load short of one that would shatter the masonry is the actual deflection anything like the deflection calculated according to the assumption that all wind is resisted by the frame.

Wherever steel is so encased in masonry as to constitute definite and permanent stiffening of the member this circumstance should be taken into account in establishing stiffness factors. It would thus be frequently necessary, as suggested by Frederick Martin Weiss,[15] Jun. Am. Soc. C. E., and

[15] *Proceedings*, Am. Soc. C. E., May, 1932, p. 961.

Mr. Julian, to accord a higher stiffness factor to girders but not to columns, than they would possess as bare members. Rudolph P. Miller, M. Am. Soc. C. E., very properly calls[16] attention to the incapacity of certain types of light floor construction to contribute materially to the rigidity of a building. Reference was made to this matter in the first progress report.

(6).—Allocation of Wind to Braced Bents.—Mr. Smith finds difficulty in assigning the wind load accurately to the various braced bents of an irregular building. This is a matter that is also being considered by the Sub-Committee.

In such allocation Mr. Weiss suggests that additional capacity for load might be assumed in case a bent is situated in a wall or partition. This would necessarily involve the crediting of walls and partitions with strength as well as rigidity, a step that is not deemed desirable.

In cases of delivery of eccentric shear to the columns of a story, instanced[17] by A. H. Finlay, Assoc. M. Am. Soc. C. E., a distribution taking proper account of the ecentricity of the total lateral load with respect to the resistance points afforded by the columns, or to the resistance lines afforded by the bents, would need to be worked out for the particular case.

(7).—Deflection and Vibration.—The method of determining the deflection of a frame due to column bending outlined in the Appendix to the second progress report was submitted as a convenient one and not necessarily as the best. Mr. Pratley has indicated an excellent alternative. Professor Finlay's misgiving as to neglect of the relation between the computed and the actual web deflections is unfounded. The results must be modified, of course, to take into account the stiffening effect of the masonry.

The method of impressing into the wind system the exterior columns not belonging to the wind bents, as suggested by Mr. Mensch, is based upon the capacity of the wall girders to act as double cantilevers projecting each way from the column belonging to a bracing bent. It is doubtful if much dependable relief can be secured from this source.

Mr. Mensch's confidence in his ability to predict deflections and the amplitude and frequency of vibration by analytical means is not shared by many of those who have given careful thought to the subject. D. C. Coyle, M. Am. Soc. C. E., doubts[18] whether such treatment of any ordinary building will give any reliable information as to its vibrations[19]. Mr. Pratley is of the same opinion. In this, the Sub-Committee concurs.

With a view to affording a more reliable basis for the forecasting of the vibrational behavior of buildings, and the reaction of the occupants thereto, certain experimental work has been outlined which will be undertaken as soon as funds are available.

(8).—Cross Method of Analysis.—Such endorsement as was given the method by Hardy Cross, M. Am. Soc. C. E., proposed in the second progress report[20] had reference to its use as a tool for the analysis of shallow bracing

[16] *Proceedings*, Am. Soc. C. E. May, 1932, p. 948.
[17] *Loc cit.*, August, 1932, p. 1119.
[18] *Loc. cit.*, p. 1100.
[19] *Journal*, Am. Concrete Inst., June, 1932, p. 679.
[20] *Proceedings*, Am. Soc. C. E., February, 1932, p. 217.

systems in typical frames of moderate height only. Application of the process to tall, slender frames had not been made to that time. It was recognized, however, that if account is taken of joint translation, it gives results that are in accordance with the principles of fundamental mechanics, including the least work relation. No suggestion has been made at any time that the method is an appropriate one for design.

Nevertheless, while in its extended form the Cross method is theoretically correct, under certain circumstances it may be unwieldy. The Sub-Committee is particularly indebted to Mr. Spurr for his comprehensive investigation of the circumstances under which the method does not effect a ready solution and the conditions under which it may be profitably used for analysis. Mr. Spurr has pointed out, as did Messrs. Weiss and Feld, that where the columns are much stiffer than the girders, the convergence on balancing is very slight, and a large number of cycles is necessary in order to accumulate sufficient moment in the girders to counteract the moment in the columns. V. A. Vanoni, Jun. Am. Soc. C. E., and Mr. M. P. White have also shown[21] that it may be necessary in the case of a bent proportioned in a haphazard manner to adjust for moments of the third and fourth order. Under the usual circumstances of wind analysis this is impracticable.

As a result of the studies thus far known to the Sub-Committee, it appears that there is no ready and practicable method of estimating the vertical translation of the joints of a frame not proportioned to maintain the joints at a floor in line under the action of lateral force. The determination of the final moments in such a frame in cases where joint translations are substantial would become very difficult. For a tall building, in which the columns, except near the top, will inevitably be much stiffer than the beams, and in which vertical joint translation near the top may be very great, it is necessary either to submit to a highly tedious analysis, or, at the outset, to proportion the frame in such a manner that the floors will remain plane after bending and the effect of vertical translations on the moments will be negligible.

If the building is very tall, even if the floors are held plane, the Cross method cannot, without modification, be utilized without considerable work. The ramifying and extended effects felt in a very tall frame when moment is applied at any one point become very difficult to follow through. For this reason Mr. Spurr's device of isolating portions, say, stories, of the frame by the introduction of hypothetical hinges merits careful consideration. Its value under various conditions is now being investigated by the Sub-Committee.

The Sub-Committee is indebted to C. M. Goodrich, M. Am. Soc. C. E.,[22] for a useful short-cut to facilitate the workability of the Cross method by adding arbitrary moments in the various stories, having regard to the extent of cancellation that occurs on the first balance. The success of this device is in large measure dependent upon intelligent guessing of the probable net yield of fixed-end moments introduced at column ends.

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[21] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1124.
[22] *Loc. cit.*, May, 1932, p. 949.

It was not assumed, as Mr. Mensch appears to think, that the moments found to exist at intersection points by the Cross method would be the moments for which beam or column sections would be proportioned. Clear spans are the obvious dimensions to use for such computations.

(9).—Arrangement of Wind Bracing in Tier Buildings.—Mr. Mensch has interpreted the Sub-Committee's report somewhat incorrectly with respect to the combination of deep and shallow bracing in the same bent. The objection was based on the difficulty of using shallow bracing efficiently in the same bent with deep bracing when the principle of equal deflections in the panels of each bent is observed.

(10).—Details of Wind Bracing.—Attention has properly been drawn by Messrs. Weiss, Wing, and Julian to the need for considering the combined effect of gravity and wind loads on the connections. This is an essential of careful designing and is presumed to be observed when designs are made according to the Sub-Committee's recommendations. Actual deformations due to connections are discussed in Division (*C*) of this report.

(*B*) Wind-Bracing Design

(1).—General.—The problem of wind-bracing involves both proportioning the members of the frame (that is, design), and the analysis of the resulting structure after its approximate make-up has been determined. Methods of procedure for determining, accurately, the moments and shears, and the stresses resulting therefrom, may not always be practicable for purposes of proportioning. Thus, while the Cross method of moment distribution is valuable for analysis under certain circumstances, it is less satisfactory for purposes of design.

In its second progress report the Sub-Committee gave some consideration to the analysis of an existing or assumed frame. In this report, it has seemed desirable to discuss the related problem of design.

In the first progress report eight definite recommendations were made which have been discussed by members of the Society. These were revised in the text of the second progress report, and were covered by Recommendations (1), (2), (3), (6), (8), (9), and (10) of the latter report. It is believed that, for the present at least, these combined recommendations form a sound basis for design. The purpose of this third progress report is to consider further the underlying principles which should control wind-bracing design, and to suggest a practical method of application. Provision of adequate wind-bracing is involved in the economical and efficient design of buildings of many types. Structures vary as to shape, size, height, weight, materials of construction, and exposure. In some, the masonry content may be large, while in others it may be a minimum. Engineers in the past have faced this problem, guided largely by experience and the custom of some particular period or locality. In this manner certain practices and certain methods of design more or less acceptable for buildings of moderate height and reasonable proportions became established. These practices and methods cannot, in general, be applied to buildings of other proportions without considerable modification.

(*2*).—*Methods of Design Considered.*—It thus appears desirable in approaching the problem of wind-bracing design to review at the outset the methods that have been used extensively in the past, and to point out their salient characteristics, with a view to defining in a general way their usefulness and limitations. An improved method of wider applicability and greater accuracy will then be outlined. Consequently, this report will deal with three methods of design. These are: (I) The portal method; (II) the cantilever method; and (III) Spurr's method.[23] Each of these methods is discussed generally in what immediately follows and in greater detail in the Appendix.

(I) Portal Method

(*3*).—*Essentials of the Method.*—Until a few years ago the portal method was probably more extensively used than any other. Its great virtue is simplicity. As commonly used, it does not concern itself with elastic relationships. In its simplest form the total wind load on a structure is divided between the bents according to their spacing, each bent taking vertical strips of wind load the width of which is defined by the sum of one-half the distance of the bents on either side. The accumulated shears based on this wind load are calculated for each story of the bent, and, for the simple case assumed, the wind shear in any story is divided equally between the panels in the bent. It is then purely a matter of producing static equilibrium for a given total load. The girder spans and their depths may be the same or different, and hence in its simplest form this method does not contemplate any relation between bending stresses and span-depth ratios. As far as possible, the connections are made alike, and the total value of the connections at any floor line is made to equal the total floor moment due to wind.

It is apparent that the distribution of horizontal wind shears is purely arbitrary. On this basis the vertical wind stresses in the columns are obtained, and the entire frame is checked for strength. From a practical standpoint many satisfactory designs have been prepared in the past by this method. The accuracy of the method may vary within wide limits, depending on the particular frame. It must be admitted that in these circumstances this method has limitations varying with the degree of accuracy desired.

(II) Cantilever Method

(*4*).—*Essentials of the Method.*—In the so-called cantilever method the shears in the various panels are distributed arbitrarily in such a manner as to produce vertical unit wind stresses in the columns that are proportional to the column distances from the neutral axis of the bent. From the resulting shear distribution the moments in all the connections and members are calculated, irrespective of the relative stiffness of the girders and columns, by assuming the points of contraflexure at their mid-spans. In its original form no attempt was made in this method to check the flexibility of the members against the design shear distribution, and the consequent capacity of the frame to act as assumed.

[23] See "Wind Bracing," by H. V. Spurr, M. Am. Soc. C. E.

(5).—*Comparison with the Portal Method.*—The resulting design may, or may not, be an improvement, from the standpoint of accuracy, in any particular case, as compared with the design that would have been obtained by using the portal method. The degree of accuracy in either case will be largely incidental, although in both cases the strength of the connections is made equal to the total wind moment, thereby producing static equilibrium. It may be safely stated, however, that the cantilever method is generally more suitable for high frames than the portal method.

(6).—*Combined Deep and Shallow Bracing.*—Frequently, in practice, considerable variations in both methods have been obtained by the introduction of knee-braces and frames at various places throughout the building, where the architecture would permit. Where this has been done, it has been customary to allocate arbitrarily a definite shear resistance at these points, simply on the basis of strength. By such methods of design a frame of considerable, although unknown, strength and stiffness may be obtained, which, taken with the strength and stiffness of the masonry content, may produce a satisfactory structure in the light of past experience.

(III) Spurr's Method

(7).—*Necessity for Considering Elastic Behavior.*—In recent years, considerable thought has been given to more exact methods of design and analysis, with particular reference to the elastic action of the various members in the frame. Columns and beams subject to bending moments bend, and members subject to direct axial stress change in length. Such actions in the frame have a direct bearing on the true shear distribution, and, consequently, on the true bending moments in the members and at the joints.

In attempting to arrive at a more accurate analysis of frame action, students of the subject were led into the development of the slope-deflection method, and, later, of the Cross method, both aiming to reach true shear distribution by the balancing of moments at joints, taking into account the relative stiffness of all members meeting at each joint. Both methods are somewhat cumbersome in application for tall frames, and to keep them at all workable, the assumption has commonly been made that the joints remain fixed in elevation and that there is no change in the length of any of the members.

In high building design the change in length of columns, due to their axial wind loads, makes this assumption seriously erroneous, and, in general, any attempt to make direct allowance for the effect of the column action renders these methods unwieldy as working tools. Unless in the preliminary design of the bent due attention has been paid to the need for establishing such elastic relationships as will lead to a practicable wind analysis, the vertical movements of the successive joints at each floor will be irregular and not readily calculable. The existing complexity is evident when it is remembered that not only do the vertical reactions of the web system upon the columns affect the movements of the joints, but the movements of the joints affect the reactions. It is true, unfortunately, that the accurate analysis of an injudiciously proportioned frame is a difficult and practically impossible task

by any other known method than that of least work. More or less "cut-and-try" methods are often the designer's best tool when confronted with such a situation. Even in these cases, however, the Cross method may effect a satisfactory analysis of the design, although its application necessarily involves considerable time and labor.

In actual wind problems the aim should be the creation of a design that is "prudently" or "correctly" proportioned. In a frame so devised the change of length of columns would have no material effect on the panel shears, or upon the web analysis generally. An elastic relationship yielding this desirable result is attained when the floors are maintained plane after bending. Incidentally, for a given spacing and composition of columns, this simple type of deformation produces the minimum deflection of the bent. Such action is possible, of course, only when the web system is proportioned and connected to the columns so as to transfer to them the vertical increments of load consistent with unit cantilever action. Consequently, the beams in the interior panels must be more rigid than those in the exterior panels.

Given a correctly proportioned frame, as defined in the foregoing paragraphs, the Cross method then becomes a readily applied tool for analysis. Both the slope-deflection and the Cross methods are more useful for analysis than for design.

(8).—Placement of Wind-Bracing.—In buildings of moderate height which are not particularly slender, there is little need for considering the wind-bracing in determining the architectural layout, as it is generally possible to adapt sufficient bracing to almost any frame which is developed. In high tower buildings, or even in buildings of moderate height that are extremely slender, some thought should be given to the bracing in the early architectural studies, so that this feature may be incorporated without upsetting the architectural plan, or without undue expense.

(9).—Requisites of Effective Wind-Bracing.—Proper design of wind-bracing involves provision for stability, strength, and rigidity. The relative importance of these three factors varies considerably in practical designs, but proper provision must be made for each.

The frame of a building is made up, in general, of vertical members (the columns), and horizontal members framed between the verticals (the girders and beams). In tier buildings the horizontal members in the floors occur at fairly regular intervals throughout the height, and with the floor-slabs form horizontal diaphragms uniting the columns at each floor level. The purpose of the wind-bracing should be to preserve this frame without undue distortion when subjected to horizontal wind forces.

(10).—Truss Action of a Bent.—In a building frame, as described, and in which no diagonal bracing is introduced, the tendency of the panels formed by the columns and their connecting beams or girders to distort, due to horizontal thrust, is resisted by the connections at the columns, and by the members in bending and direct stress. Where several columns occur in a line coinciding with the line of the thrust, they form a bent which, to a degree depending on the rigidity of the beams, their connections, and the

nature of the bracing utilized, tends to resist the horizontal thrust as a vertical truss with the columns forming multiple chords, and the columns, beams, and girders forming the web. These columns on the windward side of the axis of the bent, or truss, become tension chords, and those on the leeward side, compression chords. There is consequently a lengthening of some columns and a shortening of others in each bent as wind is resisted.

In a successful design the building must be assumed to act as a unit, and not distort appreciably in resisting the design wind load. The floors must be of sufficient strength and stiffness to act as horizontal diaphragms for distributing the pressure of one story to the bents, and if the floors are not to be distorted the bents carrying the pressure must deflect alike. This fundamental principle was emphasized in the Sub-Committee's first progress report.

(*11*).—*Effect of Walls and Partitions.*—Although the Sub-Committee has recommended that the structural effect of walls be disregarded in wind-bracing design for high buildings, it is well to recognize that low buildings, and buildings of broad base with heavy continuous walls, derive much bracing advantage from them. Where they properly may be considered as adequate, or partial, bracing, and where they should be disregarded, must be left to the individual analysis and judgment of designers. The countless variations in shape and proportion of buildings, and the varying make-up of walls, renders impossible any general rule on this point. In the remainder of this report the effect of walls and partitions in wind-bracing design will be disregarded.

A class of buildings that requires little discussion herein is that of large area with closely spaced columns and regular floor framing, in which the sum of the moment values of the ordinary beam or girder connections is ample to balance the wind moment. Although exact limits for such a class can not well be defined, they will generally include such structures as ordinary store buildings, warehouses, public schools, etc.

(*12*).—*Design Procedure.*—In the design of frames that are tall and slender, having a ratio of height to width of 4 or 5 to 1, or more, accurate analysis of the wind-bracing becomes more and more necessary as the slenderness ratio increases.

Assuming a structure of the proportions mentioned, the designer's first step should be the preparation of preliminary plans and a column schedule based on dead load and live load requirements. The next step should be the selection of vertical bents in the frame in each direction which can reasonably be expected to act in resisting the wind. If the plan lends itself to the introduction of deep bracing (in the walls, between elevator shafts, at permanent partitions, etc.), diagrams of the bents should be drawn for ease in study and the total wind load on each bent determined by the natural exposure. The relative rigidities of the several bents should next be ascertained. These rigidities, in a well-designed structure, are inversely proportionate to the deflections of the various bents under the same load arising from chord-length changes only. A redistribution of the assumed wind loads is then made if the variation in rigidities requires it.

To this point it has been assumed that the web deflection of the several bents—that is, the deflection due to the distortion of the beams and their connections and the bending of the columns—can be made alike. If their chord deflections do not vary too greatly, it may be satisfactory to leave the wind load allocation as first made, and to balance the discrepancies, by means of "tuning" the web deflection, so that the bents will give equal deflection under the assumed loading.

The analysis of the bents for chord deflection, and the subsequent web analysis, should be made on the assumption that each bent will act as a vertical truss, and that a horizontal plane through the bent will remain a plane after bending. To conform to this the distribution of the total horizontal shear in each bent must be made between the individual panels, so that the chords (the columns) will carry wind stresses proportionate to their distances from the neutral axis of the bent.

With the shears in the panels of each bent fixed as described, the web deflections of each panel should be investigated and made to agree with the principle that all bents must deflect alike (after adding chord and web deflections) and that the web deflections in all panels of the bent must be the same. This principle must hold whether deep or shallow bracing is used. It may be necessary to stiffen some panels and loosen others to accomplish this. Of course, the limits of stress for each member under the combined action of dead load, live load, and wind load must be kept in mind.

(13).—Economy.—Economy, of course, must be an aim in any design. Neglecting consideration of wind, each frame is made up of members of certain sections as required by dead load and live load considerations. An economical wind-bracing design will utilize these requisite members, and will add as little as may be to the weight and intricacy of the frame in obtaining the desired stability, strength, and stiffness. The ideal wind-bracing solution, disregarding the frame as determined by dead load and live load requirements, may or may not be the most economical when all factors are considered.

For the resistance of wind stresses only, a deep form of bracing will almost always prove economical, but as the object of the structural designer is to design his frame for all purposes in the most economical manner, it may be found that members as designed for dead load and live load only will care for the wind with little or no modification, even if uneconomical for wind duty only.

(14.)—Limiting of Deflection.—In extremely high or slender buildings the designer should consider the necessity for placing a limitation on the static deflection permitted under the design load. There is a lack of data for fixing the proper limitation with certainty. In extremely high tower design a computed deflection under assumed triangular loading of two one-thousandths of the height of the building, assuming all the wind load as absorbed by the frame, has proved workable and satisfactory to date. Limits of two or three times this value seem to give unobjectionable results in less extreme buildings. The limitation must be such as to insure not only the preservation of masonry, but also the provision of a building sufficiently rigid for com-

fortable occupancy. The Sub-Committee hopes through an experimental program, to develop further data on this phase at a later date.

A typical analysis of an assumed building frame along the lines described herein is given in the Appendix.

(*C*) Rigidity of Riveted and Welded Wind Connections

(*1*).—*General.*—Attainment of the highest degree of rigidity consistent with reasonable cost is an important objective in wind-bracing design. To this end, it is desirable to avoid the use of details which subject thin flanges of T-attachments or of split channels, or the legs of thin angles to heavy cantilever bending, or which apply large tensile or compressive forces to column flanges at a considerable distance from the junction of the latter with the web.

It has sometimes been represented that the elongation of the shafts of tension rivets is a source of a large part of the joint yield. The results of experiment [24, 25] show, however, that within the initial stress of the tension rivets, which will ordinarily be as much as 70% of the yield point of the bar stock from which the rivets are made, or greater than 26 000 lb per sq in., this is very small. As a matter of fact, the deformation of the shear rivets connecting the joint material to the beam flanges is a far more important source of flexibility.

(*2*).—*University of Illinois Tests.*—Little experimental information has been available heretofore concerning the actual angular deformation of connections such as might be used for wind resistance. The tests[26] by Wilbur M. Wilson, M. Am. Soc. C. E., and Professor H. F. Moore yield some data, but not with respect to the types of connections now considered practical or unsuitable as elements in a wind-bracing system. At loads producing a flexural stress of 24 000 lb per sq in., and rivet shear and bearing stresses of 18 000 and 36 000 lb per sq in., respectively, the following angular deformations nominally due to the connections were observed:

Type of connection	Angular deformation, in thousandths of radians
Gusset-plate replacing column web and inserted between flange angles of 24-in. girder	1.83
Gusset-plate connected by vertical angles to column flange and inserted between flange angles of 24-in. girder	1.63
Transverse-angle knuckle connections to 12-in. beam	2.80
Lug-angles running in direction of 12-in. beam	1.08
24-in. girder connected to column by vertical angles on girder web	0.68
12-in. beam connected to column flange by vertical angles on beam web	7.00

[24] *Bulletin No. 8*, Section No. 16, School of Eng. Research, Univ. of Toronto, Toronto, Ont., Canada.

[25] *Bulletin No. 210*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[26] *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

(*3*).—*University of Toronto Tests.*—Reference has been made in the second progress report to the probable superior rigidity that might be obtained in wind connections by the substitution of welding for riveting. Some experimental evidence is now available on this matter in tests conducted at the University of Toronto, Toronto, Ont., Canada, the results of which are being incorporated in a research bulletin.

Two types of welded connection and one type of riveted connection were tested. In some cases the welded connections were welded T's at the top and bottom, while in other cases they were based on the use of small triangular flanged gussets at top and bottom. The riveted connections were riveted T's, top and bottom, with the usual riveting arrangement.

These connections were attached to 18-in. 47-lb Carnegie beams and were incorporated into two general classes of double-cantilever specimens, namely (1) plate specimens, in which the connections were made to a plate representing the web of a column; and (2) column specimens, in which the connections were made to the flanges of a 12 by 12-in. H-column.

A factor of safety of between 2.25 and 2.50 was used in the design of rivets and welds in the moment-resisting part of the connections, while, in other respects, the permissible stresses of the American Institute of Steel Construction were used. Selection of the working stresses was made with a view to rendering failure of the connection welds or rivets more likely than failure elsewhere. A factor of safety of between 2.25 and 2.50 is, moreover, consistent with that obtained when wind stresses are allowed to exceed the stresses permitted for dead and live load by as much as 50 per cent. Actual factors of safety by test ranged from 1.44 for a welded T-specimen to 2.47 for a gusset-plate type of welded specimen with stays welded between the column flanges.

The results of the investigation cited warrant the following observations:

(*a*) The story drift angle due to the connections is much less than that due to the deformation of the plain beam ideally connected with a connection of zero length.

(*b*) Due to the large deformations that accompany the application of load to the unstayed flanges of a column at any appreciable distance from their junction with the web, connections to column flanges give rise to more drift than connections to column webs. Stiffening of column flanges by welding a stay between them reduces the disparity.

(*c*) Whether the connection is to the web or to the flange of a column, welded connections are stiffer than riveted T's for the same orientation of column.

(*d*) Welded T's are not as stiff as the built-up gusset-plate type of welded connections.

(*e*) Welded connections of the gusset-plate type to the web of a column may, by reason of the flange reinforcement effect, give a stiffer bay than exists with an ideally connected beam of uniform moment of inertia.

(*f*) For wind in one direction, the drift angle due to the connection itself varied, for the capacity moment on the critical connection, from −0.00017

radian for the gusset-plate type of welded connection to a column web to +0.0024 radian for the riveted connection to a column flange. The maximum for a welded connection was +0.00060 radian for a welded T-connection to a column flange. These may be related, for purposes of rough comparison, to the mean values of +0.00183 and +0.00163 radian obtained by Professors Wilson and Moore for corresponding loads on two types of riveted gusset-plate connections.

Conclusions

The conclusions reached by the Sub-Committee may be summarized as follows:

(1) Whatever prescribed wind force is finally adopted must be based not only upon recorded wind velocities at observation stations, but also upon the modifying influences of shielding and turbulence and the general conditions set up for proportioning the frame and its details. It continues to be the belief of the Sub-Committee that these modifying influences and conditions make the wind force tentatively recommended in the first progress report—that is, 20 lb per sq ft for the first 500 ft, increased above that level at the rate of 2 lb per sq ft for each 100 ft of height—generally adequate for the United States and Canada.

(2) Considering the relative probability of the occurrence of full specified wind load and the simultaneous occurrence of full specified wind load and full specified live load, it is believed that the ratios of permissible stresses recommended in the second progress report for members carrying various percentages of their total stress in the form of wind, are satisfactory. Under any reasonable wind overload, the factor of safety would be approximately the same for members carrying a large percentage of wind as for those carrying a small percentage.

(3) The use of higher permissible stresses for rivets and bolts is believed to be justified by the fact that there is a greater real factor of safety for these parts than for members subjected to comprehensive buckling.

(4) A factor of safety of 1.5 for stability is believed to be adequate in that the resistance is based on gravity and not on the resistance of materials which may vary in strength, and since the probability of zero live load existing with the maximum wind load is small.

(5) The stiffness factors for members permanently encased with concrete and to which it is thoroughly bonded should be calculated with proper allowance for the effect of the encasement. As far as wind distribution and resistance are concerned, reliance should not be placed upon floor construction, light or heavy, unless analysis shows it to be adequate for the duty.

(6) Allocation of wind load to braced bents should take into consideration the eccentricity of the total lateral load to the lines of resistance.

(7) Vibrational characteristics of buildings and the relation of sensation thereto must be determined experimentally before buildings can be designed with certainty to yield a desired degree of comfort to the occupants.

(8) The Cross method of analysis is satisfactory for shallow bracing systems in buildings of moderate height in which the frame has been pre-

viously proportioned so as to maintain the floors substantially plane. It may become unwieldy, however, for tall structures, particularly if the design has not been prepared with a view to holding the joints at any floor of a bent in a straight line. Even in cases where the design has been made so as to render it unnecessary to compute the actual vertical translation of joints, it is desirable to isolate stories and apply the Cross method to them individually, adding to the direct stresses in the columns the stresses in the columns immediately above.

(9) If care is exercised in proportioning, the portal method of design may be used with satisfaction for buildings of moderate heights and proportions. When the beam spans are equal, and the beam stiffness factors are also equal, the method is accurate except for (*a*) the effect of column bending as a factor in web distortion; and (*b*) the effect of change of length of the columns under axial wind loads.

(10) The cantilever method is generally more suitable for high frames than the portal method. With equal beam spans and equal beam stiffness factors, the method is accurate, except for (*a*) the effect of column bending as a factor in web distortion; and (*b*) the effect of unequal drift from beam bending under the assumed shear distribution.

(11) Mr. Spurr's method offers a simple means of proportioning a frame to ensure that the floor joints will remain in a straight line. Analysis of a frame thus proportioned is greatly facilitated by reason of the negligible influence of vertical translation of the joints. The method is based upon the assumption that the web system in every panel may be made rigid enough to transfer to the columns at design stresses the vertical increments of wind stress that should be delivered to them if the bent is to act as a simple cantilever.

(12) Experimental evidence indicates that typical welded wind connections are stiffer than typical riveted T-attachments for the same orientation of the columns. By reason of the flange reinforcement effect, welded connections may give a stiffer bay than exists with an ideally connected beam of uniform moment of inertia.

Respectfully submitted,

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Francis P. Witmer.

January 6, 1933.

APPENDIX

(I) Portal Method

(*1*).—*General.*—The panel shears, floor moments, and moments in the connections as determined by the simplest form of the portal method are indicated in Fig. 2. In accordance with this, the panel shears are equal, the points of inflection are at mid-span of the beams and at mid-height of the columns, and axial wind stresses exist in the exterior columns only.

(2).—*Attainable Accuracy.*—When the beam spans and the moments of inertia of the beams are alike the portal method is accurate except for (*a*) the effect of column bending as a factor in web distortion; and (*b*) the effect of change in length of the columns under axial wind loads. Generally speaking, the effect of column bending is relatively larger in a low building than

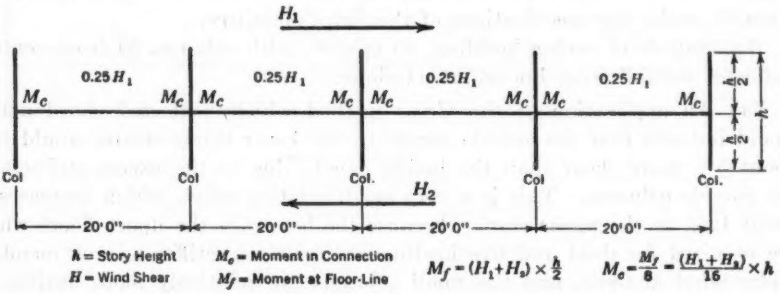


FIG. 2.—SIMPLE PORTAL METHOD.

in a high one, because the columns are relatively more slender and they contribute materially to the web distortion. Therefore, in a low building with large story heights material errors would result from the effect of column bending in the example illustrated by Fig. 2, unless the interior columns were of the same section, and the exterior columns were approximately one-half as stiff as the interior columns. If the moments of inertia of the outside columns were exactly one-half the moments of inertia of the inside columns, then with equal beam spans and beams of equal moments of inertia the portal method would be correct for a low building, in which chord action is unimportant. The method would also be approximately correct in a low building with these column relations and with unequal spans between columns, provided the extreme fiber stresses in the beams under bending from wind were inversely proportional to their span-depth ratios.

For these reasons, the portal method has great usefulness, and with some care in proportioning the beam sizes it is suitable in buildings of moderate height and proportions. For special frames of irregular character a more definite design procedure may be advisable, which will take into account more accurately the relative stiffness of the various members in the building.

In an office building of normal construction, with columns about 20 ft on centers, the exterior columns will average about 75% as stiff as the interior columns. In the simple portal method it is assumed that the exterior columns take only one-half the shear taken by the interior columns. Therefore, the exterior columns would be 50% more rigid than they should be to suit the shear distribution assumed. With equal beams on equal spans this excess stiffness in the exterior columns would tend to increase the shear taken by the outside panels, and the Cross method would reveal this effect in a low building in which the columns are relatively flexible and the chord effects are negligible.

However, in a high building, the chord action would overbalance the web effect of the extra stiff outside columns, and the inside panels would actually carry more shear than the outside panels. Therefore, in such a building an analysis by the Cross method which neglected chord action would be less accurate than the simple portal method in many cases. This is revealed in a study of a 40-story office building of normal construction designed for strength under the specifications of the Sub-Committee.

An analysis of such a building, 80 ft wide, with columns 20 ft on centers, indicates the following important relations:

(*a*) An application of the Cross method which neglected chord action would indicate that the outside panels in the lower thirty stories would take about 3% more shear than the inside panels, due to the excess stiffness in the outside columns. This is a very small relative effect, which increases to about 12% in the upper stories, because the beams in the upper floors which are required for dead and live loading are relatively stiffer as web members under wind analysis, and the small columns are relatively more flexible, so that the columns participate more generously in the web distortion in the upper stories. Thus, the 50% excess stiffness in the outside columns has a larger influence in the upper floors on the distribution of the horizontal shear than in the stories below.

(*b*) The effect of chord action is in the opposite direction, and completely overshadows the considerations in Relation (*a*). If the vertical stresses (consistent with the distribution of wind shear by the portal method) occurred in the outside columns, it would result in a reduction of shear resistance in the outside panels at the top of the building of about 150% (due to the accumulated change in length in the outside columns only). This is an inconsistency and not an actual percentage of error. However, the actual error would be not only material, but of a varying amount at the different levels, and very difficult to determine. It is not practicable to embark on such an involved design analysis. Either the engineer should make up his mind to accept material errors of unknown amount, or he should follow a procedure that will largely eliminate the complications of analysis.

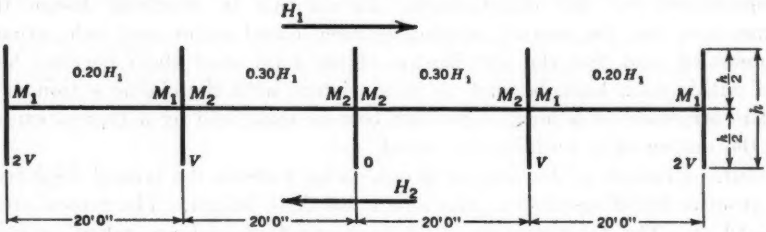
(II) Cantilever Method

(*3*).—*General.*—The panel shears and connection moments as determined by the simplest form of cantilever theory are indicated in Fig. 3. In accordance with this method the total axial wind stresses in the columns, regardless of the column areas, are proportionate to the distances from the neutral axis of the bent. The points of inflection are at mid-span of the beams and at mid-height of the columns.

(*4*).—*Attainable Accuracy.*—When the beam spans and the moments of inertia of the beams are alike this method is accurate except for (*a*) the effect of column bending as a factor in web distortion; and (*b*), the effect of unequal drift from beam bending under the assumed shear distribution.

The effect of column bending as a factor in web distortion reduces with the increasing height of the building, since the columns become relatively

stiffer with the number of floors supported by them. Material errors may result, however, if the beams are not properly proportioned as to stiffness for the assumed bending moments. Fairly accurate results may be obtained in design if the beam sizes are chosen so that under the assumed bending



$M_2 = 1\frac{1}{2} M_1$ $\quad 4(M_1 + M_2) = \frac{(H_1+H_2)h}{2}$ $\quad M_1 = \frac{(H_1+H_2)h}{20}$ $\quad M_2 = 3\frac{(H_1+H_2)h}{40}$

FIG. 3.—SIMPLE CANTILEVER METHOD.

moments the extreme fiber stresses in bending under wind vary inversely as the span-depth ratios of the beams.

With irregular column spacing, reasonable accuracy in design will depend largely upon the care exercised in the choice of beam sizes to meet the conditions of equal drift from beam bending in all panels.

In many cases, where approximate methods of design appear warranted by past experience, the designing engineer may desire to take advantage of an opportunity to use some deep connections, such as knee-braces, gusset connections, frames, or diagonals, in certain panels. These connections will materially reduce the bending moments in the columns and girders in the panels in which they occur; and, in general, they will tend to make such panels less flexible than those in which only shallow connections are used. These effects may require investigation and evaluation in order to maintain the degree of accuracy desired in the particular design.

A detailed discussion of the various factors that contribute to web distortion or panel drift properly belongs with an explanation of the design procedure for high slender towers which follows. Extreme precision is one part of the design, but it is neither logical nor of particular practical value.

(III) SPURR'S METHOD

(*5*).—*Elastic Behavior the True Basis.*—In an elastic structure of the multi-story type that is subjected to horizontal wind forces, the true moments and shears in the various members are dependent upon the relative rigidities of all the members, if they are properly connected. Of course, inelastic action, or slip in the connections, will affect the distribution of stress throughout the structure, and such effects are to be guarded against as far as possible by care in the design of the connections. Elastic action in the connections, whether deep or shallow, or in knee-braces or frames, is unavoidable, and quite properly such effects should be ascertained and evaluated in the design.

In general design procedure it is customary to calculate the moments, shears, and direct stresses in members from the loads and then to proportion

the members to suit the conditions of loading. This general procedure may be followed in wind design with equal accuracy, provided the fundamentals of elastic proportion are not violated. Thus, the total horizontal wind shear may be distributed definitely throughout the structure, and the latter then proportioned for this distribution. To do this in practical design it is imperative that the mutual relation between chord action and web action be understood, and that the distribution of the total wind shear between bents, and within each bent, be such as is consistent with the elastic action of the entire structure as a unit. This can best be explained by a typical example of the design of a building for wind.

(*6*).—*Example of Design for Wind.*—Fig. 4 shows the typical floor layout in an office building of 83 stories and 1 000 ft in height. The typical stories are 12 ft. The total average dead load and live load are taken as 175 lb

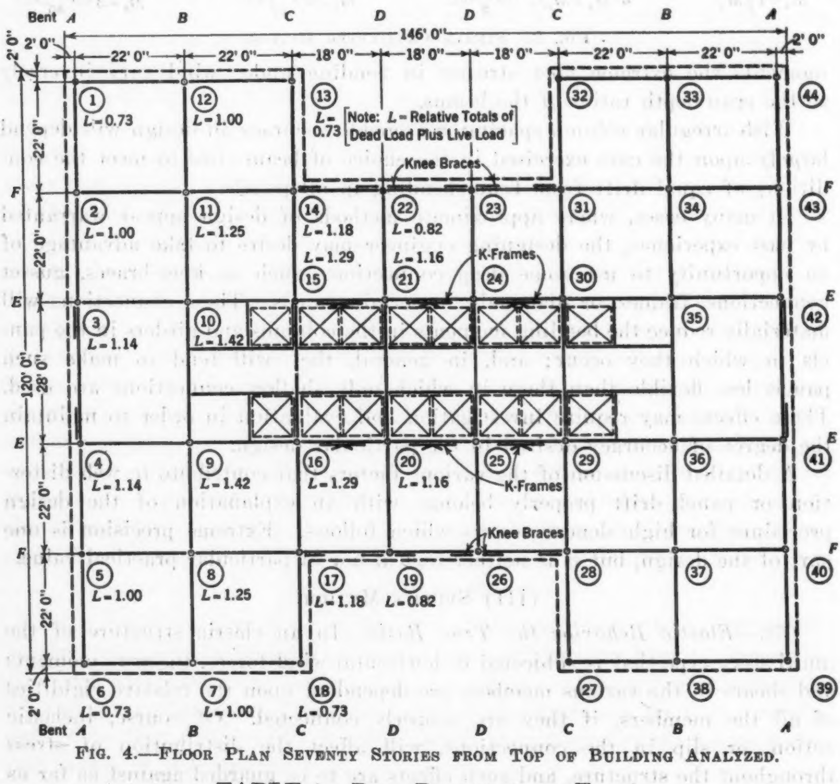


FIG. 4.—FLOOR PLAN SEVENTY STORIES FROM TOP OF BUILDING ANALYZED.

TABLE 1.—DATA PERTAINING TO FIG. 4

Description	Bents: A	B	C	D	Total
Factor of strength	137.0	181.5	146.0	44.5	509.0
Wind load: Percentage on bent	13.5	18.0	14.0	4.5	50.0
Horizontal shear, in kips	331	441	343	110	1 225

per sq ft of floor, which includes the weight of the steel frame and fire-proofing. The exterior walls are assumed to weigh 85 lb per sq ft of gross wall area. For simplicity, the wind force has been assumed as 20 lb per sq ft throughout the height of the building. Given these units, the spacing of columns, center to center, and the outside dimensions of the tower, the

TABLE 2.—DEAD LOADS PLUS LIVE LOADS APPLIED TO COLUMNS AT ANY ONE FLOOR AND RATIOS OF DEAD LOADS PLUS LIVE LOADS (COLUMN RATIOS) FOR ALL COLUMNS

Column Nos.	Determination of load, in pounds		Total dead load, plus live load, in pounds	Column ratios
2, 5, 7, 12	Floor, (22 × 11) + 22 = 264 sq ft at 175 lb per sq ft Wall, 22 at (85 × 12)	= 46 000 = 22 000	68 000	1.00
1, 6, 13, 18	Floor, (11 × 11) + 23 = 144 sq ft at 175 lb per sq ft Wall, (22 + 3) at (85 × 12)	= 25 000 = 25 000	50 000	0.73
3, 4	Floor, (25 × 11) + 25 = 300 sq ft at 175 lb per sq ft Wall, 25 at (85 × 12)	= 52 500 = 25 000	77 500	1.14
14, 17	Floor, (2 × 121) + (9 × 11) + 20 = 361 sq ft at 175 lb per sq ft Wall, 17 at (85 × 12)	= 63 500 = 17 000	80 500	1.18
19, 22	Floor, (18 × 11) + 18 = 216 sq ft at 175 lb per sq ft Wall, 18 at (85 × 12)	= 38 000 = 18 000	56 000	0.82
8, 11	Floor, (22 × 22) = 484 sq ft at 175 lb per sq ft	= 85 000	....	1.25
9, 10	Floor, (22 × 25) = 550 sq ft at 175 lb per sq ft	= 96 500	....	1.42
15, 16	Floor, (20 × 25) = 500 sq ft at 175 lb per sq ft	= 87 500	....	1.29
20, 21	Floor, 18 × 25 = 450 sq ft at 175 lb per sq ft	= 78 700	....	1.16

column loads and their sectional areas may be computed. Bents *A*, *B*, *C*, and *D* are indicated symmetrically arranged in plan, and they are individually symmetrical about axes in a common plane.

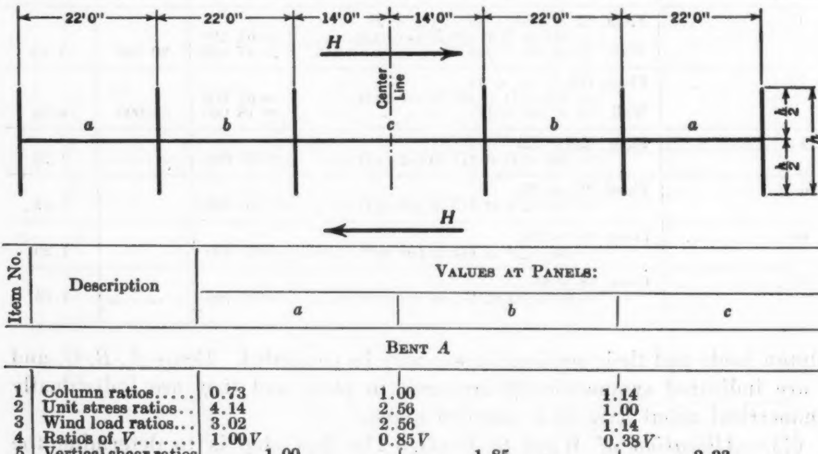
(7).—*Allocation of Wind to Bents.*—The first step is to determine the allocation of wind load to the various bents. Their relative capacity to absorb wind will be directly proportional, of course, to their rigidities. Since the required sectional areas of the columns are, in effect, known, once the combined dead and live loads have been determined, it is convenient to base the calculation of relative rigidities on chord action alone, assuming that a plane remains a plane after bending. This rests upon the knowledge that the deflection due to web action may be controlled so as to bear a constant relation to that caused by chord action and that, consequently, the relative deflections of a number of bents will be in proportion to their chord deflections. The assumption that the floors remain plane pre-supposes that the web system is proportioned so as to deliver to the columns, as chords, the vertical increments of stress that should go to them.

Table 2 indicates the computation of the column ratios; that is, the ratios of dead loads plus live loads, or the ratios of the areas required to resist these loads.

It is next necessary to determine the relative factors or indices of rigidity for the various bents. This may be conveniently done by finding that base width, or equivalent base[23] which (on the assumption that only the outside columns take axial wind stress) would give the same moment of resistance as the actual bent acting as a cantilever with unit wind stresses in the columns proportionate to their distances from the neutral axis, and then multiplying this equivalent base by the column ratio, or area factor of the column. The product is the factor of strength (see Table 1), and since the deflections are proportionate to unit stresses, it is also a factor of rigidity.

Table 3 illustrates the calculations of the relative rigidities of Bents *A* and *B* (Fig. 4). Items Nos. 1 and 7 are column ratios; that is, the ratios of total dead loads plus live loads for the various columns. If a plane remains

TABLE 3.—PROPERTIES OF BENTS *A* AND *B*, FIG. 4, AND DISTRIBUTION OF SHEARS



Item No.	Description	Values at Panels:					
			a		*b*		*c*
	Bent *A*						
1	Column ratios.....	0.73		1.00		1.14	
2	Unit stress ratios..	4.14		2.56		1.00	
3	Wind load ratios...	3.02		2.56		1.14	
4	Ratios of V......	1.00V		0.85V		0.38V	
5	Vertical shear ratios		1.00		1.85		2.23
6	Panel shears......		0.1175H		0.2175H		0.33H
	Bent *B*						
7	Column ratios.....	1.00		1.25		1.42	
8	Unit stress ratios..	4.14		2.56		1.00	
9	Wind load ratios...	4.14		3.20		1.42	
10	Ratios of V......	1.00V		0.78V		0.34V	
11	Vertical shear ratios		1.0		1.78		2.12
12	Panel shears......		0.12H		0.215H		0.33H

a plane after bending the unit stress ratios will be as in Items Nos. 2 and 8. Items Nos. 3 and 9 indicate the relative axial wind loads on the columns, these being the product of the corresponding values in Items Nos. 1 and 2, and in Items Nos. 7 and 8. Putting an exterior column on the basis of unity, or assuming the vertical wind load carried by it as *V*, the vertical

wind loads carried by all the columns in the bent will be as in Items Nos. 4 and 10. Since the increments of vertical loads in the columns are put into the latter by the vertical shears in the beams, the vertical shears in the panels will be related as in Items Nos. 5 and 11. The allocation of horizontal shears to the various bays or panels is indicated in Items Nos. 6 and 12, the method of computing shear fractions (for Bent A) being as follows:

Moment of resistance, $M_r = 116V + (0.85V \times 72)$
$+ (0.38V \times 28)$ .... $= 187.6V$
Equivalent base .... $= 187.6$ ft
Factor of strength $= 187.6 \times 0.73$............. $= 137$

$$\text{Horizontal shear to Panel } a = \frac{22}{187.6} \times H \ldots\ldots = \quad 0.1175H$$

$$\text{Horizontal shear to Panel } b = \frac{22}{187.6} \times 1.85 \times H = \quad 0.2175H$$

$$\text{Horizontal shear to Panel } c = \frac{28}{187.6} \times 2.23 \times H = \quad 0.33H$$

Similarly, for Bent B, the moment of resistance is $181.5V$; the equivalent base is 181.5 ft; the factor of strength is 181.5; and the horizontal shears are: To Panel a, $0.12H$; to Panel b, $0.215H$; and to Panel c, $0.33H$.

Computations similar to those indicated for Bents A and B having been made also for Bents C and D, the determination of the percentage of total wind on the building resisted by any one bent may be made on the basis of the factor of strength for that bent. These factors for Bents A, B, C, and D being, respectively, 137.0, 181.5, 146.0, and 44.5, the fraction of the total wind taken by Bent A is:

$$\frac{137}{2\,(137.0 + 181.5 + 146.0 + 44.5)} = 0.135$$

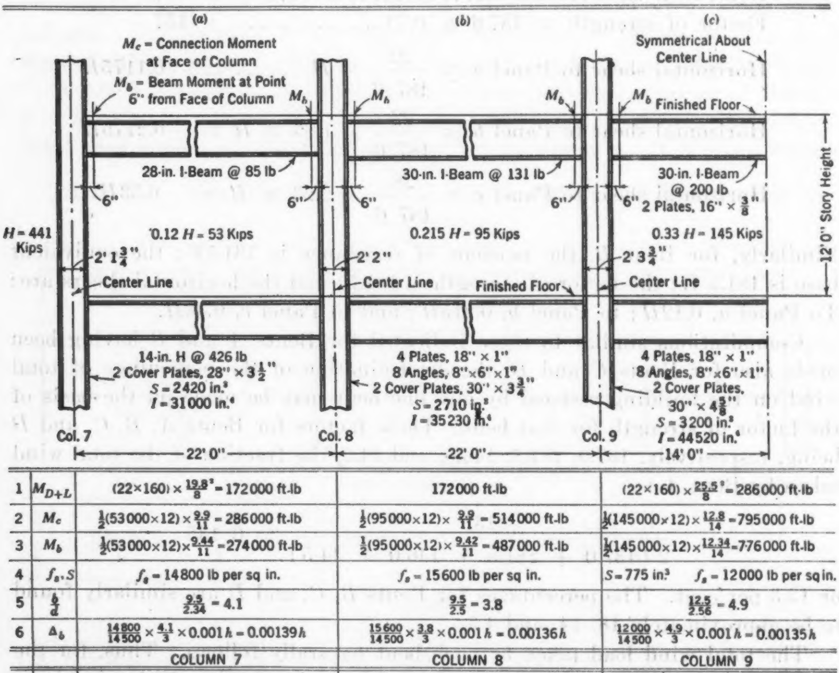
or 13.5 per cent. The percentages for Bents B, C, and D are similarly found to be approximately 18, 14, and 4.5.

The total wind load taken by each bent naturally follows. Thus, for the building under consideration the total wind load above a floor 70 stories down from the top on the 146-ft face of the building, is $70 \times 12 \times 146 \times 20 = 2\ 450\ 000$ lb. Applying the previously determined percentages, Bents A, B, C, and D will absorb 331, 441, 343, and 110 kips, respectively. These bent wind loads are indicated in Table 1.

(*8*).—*Panel Wind Shears.*—The individual panel shears for each bent having been found, as explained in connection with Table 3, the wind shear in any panel of any bent at any floor may readily be obtained. Thus, there is a definite distribution of wind load which requires a definite elastic relationship between the various members throughout the structure. This relationship will be correct if under this load distribution the web system will deflect an equal amount in all panels in any one story. The deflection or drift per story in the web system is made up of flexure in the columns and beams and elastic deformation in the connections.

(9).—Panel Drift.—In Table 4 calculations are made for Bent *B* at 70 stories down from the top where the bent shear is 441 kips, with a view to obtaining the proper beam sizes in each panel to maintain a uniform drift. From the known shears in the various panels (Tables 3 and 4) the moments in beams and columns may be readily estimated, assuming the points of inflection in the columns at the center of their clear spans, and also in the

TABLE 4.—CALCULATIONS FOR UNIFORM DRIFT IN EACH PANEL OF BENT *B*, SEVENTY STORIES FROM THE TOP



1	M_{D+L}	$(22\times160)\times\frac{19.8^2}{8} = 172000$ ft-lb	172000 ft-lb	$(22\times160)\times\frac{25.5^2}{8} = 286000$ ft-lb
2	M_c	$\frac{1}{2}(53000\times12)\times\frac{9.9}{11} = 286000$ ft-lb	$\frac{1}{2}(95000\times12)\times\frac{9.9}{11} = 514000$ ft-lb	$\frac{1}{2}(145000\times12)\times\frac{12.8}{14} = 795000$ ft-lb
3	M_b	$\frac{1}{2}(53000\times12)\times\frac{9.44}{11} = 274000$ ft-lb	$\frac{1}{2}(95000\times12)\times\frac{9.42}{11} = 487000$ ft-lb	$\frac{1}{2}(145000\times12)\times\frac{12.34}{14} = 776000$ ft-lb
4	f_s, S	$f_s = 14800$ lb per sq in.	$f_s = 15600$ lb per sq in.	$S = 775$ in.³ $f_s = 12000$ lb per sq in.
5	$\frac{b}{d}$	$\frac{9.5}{2.34} = 4.1$	$\frac{9.5}{2.5} = 3.8$	$\frac{12.5}{2.56} = 4.9$
6	Δ_b	$\frac{14800}{14500}\times\frac{4.1}{3}\times0.001h = 0.00139h$	$\frac{15600}{14500}\times\frac{3.8}{3}\times0.001h = 0.00136h$	$\frac{12000}{14500}\times\frac{4.9}{3}\times0.001h = 0.00135h$
		COLUMN 7	COLUMN 8	COLUMN 9
7	M_{col}	$26500\times4.84 = 128000$ ft-lb	$(26500+47500)\times4.84 = 357000$ ft-lb	$(47500+72500)\times4.75 = 570000$ ft-lb
8	f_s	635 lb per sq in.	1580 lb per sq in.	2130 lb per sq in.
9	$\frac{b}{d}$	$\frac{4.84}{2.15} = 2.25$	$\frac{4.84}{2.16} = 2.24$	$\frac{4.75}{2.31} = 2.05$
10	Δ_{col}	$\frac{635}{14500}\times\frac{2.25}{3}\times0.001h = 0.000033h$	$\frac{1580}{14500}\times\frac{2.24}{3}\times0.001h = 0.000081h$	$\frac{2130}{14500}\times\frac{2.05}{3}\times0.001h = 0.0001h$
11	Total Δ_{web}	$0.00142h$	$0.00144h$	$0.00145h$

beams at approximately the centers of the clear spans of the beams. Generally, it will be found that the participation of the columns in the web distortion will vary in relative amount in the different panels; but the average column participation is usually so small, as compared with the yielding of the beams, that the adjustment is made in the beams by the location of the points of zero moment at a slight distance from the mid-span points. The computations illustrate this relationship.

The first step is to determine the depths of beams to be used to provide for the moments in beams and connections. The sum of the design moments in beams, columns, or connections will not equal the total moment obtained by multiplying the wind shear by the distance between floors, because of the actual dimensions of the members. For instance, at any joint a column is restrained by the combined moments in the connections effective plus a couple produced by the vertical reactions from the beams acting at the column faces.

Similarly, the clear spans, only, of members contribute to their flexibility. Therefore, the next step is to determine the span-depth ratios of beams and columns based on their clear spans. The relative participation of the different columns in web distortion may be estimated, and it will be found of greater amount for each column in order proceeding in from the wall columns toward the axis of the bents. This effect can be provided for in proportioning the beams, as follows: The unit stresses in the beams under wind bending should vary inversely as their respective span-depth ratios to produce equal drift from beam bending only.[27] Thus, in Table 4, for this condition these unit stresses would have been 14 800, 15 800, and 12 300 lb per sq in. for the outside panels, the intermediate panels, and middle panel, respectively. The stresses used, however, were 14 000, 15 600, and 12 000 lb per sq in., respectively, in order to allow for unequal column flexure. Some slight variation in the value of total drift between panels is often reasonable, because of commercial beam sizes. It should be noted that generally the columns are maintained as determined for dead load and live load only.

The computations in Table 4 involve the determination for the three panels, or for the columns adjacent to them, of the following quantities:

Item (1).—Moments in the beams due to combined dead and live load, on the basis of simply supported spans equal approximately to the clear distance between columns. Reduction of live load for the girders brings the total load down from 175 to 160 lb per sq ft.

Item (2).—Wind moments in connections at the face of columns.

Item (3).—Wind moments in beams taken 6 in. outside column faces to allow for reinforcing effect of connection material.

Item (4).—Beam sizes and extreme fiber stresses allowed in order to produce equal drift from beam bending only and to allow for unequal column flexure.

Item (5).—Ratio of one-half the effective beam span to depth.

Item (6).—Drift from beam flexure only. This is proportional to f and to $\frac{b}{d}$. Since for $f = 14\ 500$ lb per sq in. and $\frac{b}{d} = 1$, $\Delta_b = \frac{h}{3\,000}$, the drifts per panel for the fiber stresses and for one-half the span-depth ratios listed naturally follow.[27]

Item (7).—Wind moments in columns, allowing for beam depths.

Item (8).—Flexural stresses in columns from wind only.

[27] "Wind Bracing," by H. V. Spurr, M. Am. Soc. C. E., p. 40.

Item (9).—Ratios of one-half the clear column spans to column width.

Item (10).—Drift from column flexure only, calculated similarly to drift from beam flexure.[27]

Item (11).—Total web deflection, being the sum of Items (6) and (10).

It should be noted that the beam between Columns 8 and 9, Fig. 4, is the critical one in the system, since it has the highest working stress under wind. If a stress of 18 000 lb per sq in. were used in this panel, the bending stresses in the other beams would be modified accordingly to obtain an equal drift in all panels.

For simplicity in calculations a uniform wind load of 20 lb per sq ft on the full height of the building has been used in this illustration. If the wind loads tentatively recommended by the Sub-Committee in its first progress report were used, and the beam sizes were unchanged, the working stresses in beams and columns would be increased.

The question of the amount of rigidity to secure in a particular problem is a matter of judgment. Where rigidity is deemed important, economy in the structural frame will be obtained by the use of deep beams of lightest weight consistent with requirements of strength, in combination with deep braces and frames in the panels where the shears are largest. Where knee-braces or frames are used their influence on the rigidity of the web system must be calculated, and the members proportioned to secure equal drift in all panels, on the basis of the horizontal shear distribution previously described.

In Table 4 only shallow connections have been assumed, and it is expected that they would be designed so that their elastic contribution to web drift

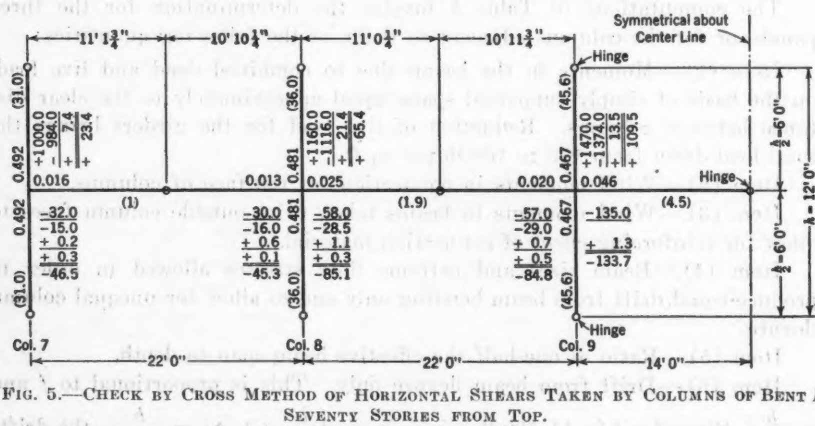


FIG. 5.—CHECK BY CROSS METHOD OF HORIZONTAL SHEARS TAKEN BY COLUMNS OF BENT *B*, SEVENTY STORIES FROM TOP.

would be of the same amount in all panels. It may be that this contribution is of a negative value, due to the local reinforcement of the beams at their ends. These refinements in design naturally follow the first design set-up, and their use depends upon the accuracy desired in the design analysis. The question of combined moments from dead load, live load, and wind load should receive attention in the design of connections, braces, and frames.

(*10*).—*Check by Cross Method.*—The accuracy of the design shown in Table 4 may be checked by the slope-deflection method, by the original Cross method, or by the adaptation of the Cross method proposed by Mr. Spurr.

Such a check for Bent *B* by Mr. Spurr's adaptation is shown in Fig. 5, proving that the true moments and the moments used in the design agree. As only relative moments are required in Fig. 5, a fixed-end moment of 1 000 (in any unit) is applied at the end of the most flexible column and

TABLE 5.—RELATIVE STIFFNESS FACTORS FOR BENT *B*, SEVENTY STORIES FROM TOP

Member	Moment of inertia, I, in inches[4]	Clear span, l, in inches	Stiffness factors, $\frac{I}{l}$	Relative stiffness factors
Beam 7–8	3 075	238	12.9	1.0
Beam 8–9	5 738	237	24.2	1.9
Beam 9–10	12 153	308	39.5	3.0
Column 7	31 000	116	267	20.7
Column 8	35 230	114	310	24.0
Column 9	44 520	114	392	30.4

larger fixed-end moments, in proportion to column stiffness factors, are applied at the ends of the other columns. The relative stiffness factors (shown in parentheses) are derived from Table 5, increasing the values therein given for the columns and for Beam 9–10 by 50% to take account of the fact that these members are assumed as pivoted at mid-length. Relative column moments and column shears obtained after two cycles are indicated in Fig. 5. These ratios correspond to the relative column shears found by Mr. Spurr's method of design and indicated in Table 6. The

TABLE 6.—HORIZONTAL SHEARS TAKEN BY COLUMNS, THE POINTS OF INFLECTION OF BEAMS BEING ASSUMED AS AT MID-SPAN; BENT *B*, SEVENTY STORIES FROM TOP

Column Nos.	Horizontal shear taken by Column, H_c, in pounds	Ratio of column shears
7	$\frac{1}{2} \times 53\ 000 = 26\ 500$	1
8	$\frac{1}{2} \times (53\ 000 + 95\ 000) = 74\ 000$	2.8
9	$\frac{1}{2} \times (95\ 000 + 145\ 000) = 120\ 000$	4.6

coefficients near the joints (Fig. 5) are distribution factors. The ratios of column moments and of column shears are computed as follows:

For Column 7: $\frac{23.4}{23.4} = 1.0$; for Column 8: $\frac{65.4}{23.4} = 2.8$; and, for Column 9: $\frac{109.5}{23.4} = 4.7$. The ratios of horizontal shears taken by columns correspond to the horizontal shear distribution of design.

In Fig. 5 the slight movement of the point of inflection in the beams from mid-span is clearly indicated, being $1\frac{3}{4}$ in. toward Column 8 in the outside panel, and $\frac{1}{4}$ in. toward Column 9 in the intermediate panel. The points of inflection move toward the more flexible of the two columns which the beams connect. In this manner the elastic adjustment is made in the beams

to take care of the unequal contribution to web deflection made by the columns alone. Thus, as indicated in Table 7, the contribution of Column 7 is 0.000033 h; for Column 8, it is 0.000081 h; and, for Column 9, this contribution is equal to 0.0001 h. The ratio of these values is 1: 2.4: 3.1, which means that the flexure of Column 9 is 3.1 times that of Column 7.

TABLE 7.—RATIOS OF COLUMN SHEAR ÷ COLUMN I AND RATIOS OF STORY DRIFT DUE TO COLUMN FLEXURE IN TERMS OF STORY HEIGHT, h; BENT B, SEVENTY STORIES FROM TOP

Column Nos.	Column shear, H_c, in pounds	I of column, in inches⁴	$\frac{H_c}{I}$	Ratios of $\frac{H_c}{I}$	Drift due to column deflection, Δ (Column)	Ratios of Δ (Column)
7........	26 500	31 000	0.86	1.0	0.000033 h	1
8........	74 000	35 230	2.1	2.4	0.000081 h	2.4
9........	120 000	44 520	2.7	3.1	0.000101 h	3.1

It is interesting to note, however, that seventy stories down in a high building this great difference in column flexure is a minor matter in the analysis, as shown by the slight movement of the points of inflection in the beams, as indicated in Fig. 5. This is because the column flexure plays such a small part in the total drift in the web system when the columns carry many floors and are relatively very stiff.

From a design standpoint it would be a great waste of material, or loss of usable space in the building from enlarged column dimensions, or both, to attempt to "juggle" with the column sizes in any way to equalize their effect in flexure. It would serve no useful purpose and would also confuse and upset the design procedure by changing the design set-up, which is based on column areas demanded by live and dead load requirements only.

(11).—Column Stresses and Stability.—The direct stress due to wind in Column 7 at the bottom of the building may be computed, as follows:

Width of building served by Bent B = 0.18 × 146 = 26.3 ft.
Wind force absorbed by bent = 26.3 × 20 = 526 lb per ft of height.
Moment at base = $\frac{1}{2} \times 526 \times (1\,000)^2$ = 263 000 000 ft-lb.
Equivalent base = 181.5 ft.
Area of column at bottom = 386 sq in.

$$\text{Axial stress in steel due to wind, } f_s = \frac{263\,000\,000}{181.5 \times 386} = 3\ 760 \text{ lb per sq in.}$$

$$\text{Ratio of height to actual width} = \frac{1\,000}{116} = 8.65.$$

$$\text{Chord deflection, } \Delta_c = \frac{3\,760}{4\,500} \times \frac{8.65}{10} \times 0.001\,h = 0.00072\,h.$$

The stress, 3 760 lb per sq in., is the maximum for any column in the building in this direction. It is the same for all outside columns in Bents A, B, and C, by design. The chord deflections in all bents—A, B, C, and D—are equal by design, and the calculations show this to be 0.00072 h.

Since the dead load in the columns will produce a unit stress of about 12 000 lb per sq in., the maximum uplift due to wind will be only about one-

third the dead load in the outside columns for a uniformly distributed 20-lb wind load, and only about 50% of the dead load for a uniformly distributed wind load of 30 lb per sq ft; therefore, the requirements of stability are fulfilled.

In Table 8 calculations are also made to determine the wind stresses in the columns at the thirteenth floor. It should be noted that the bending

TABLE 8.—WIND STRESSES IN COLUMNS AT THIRTEENTH FLOOR, IN POUNDS PER SQUARE INCH

Column Nos.	Direct stress	Bending stress	Combined stress
7..................	$\frac{70}{83} \times 3\ 760 = 3\ 170$	635	3 805
8..................	$\frac{256}{414} \times 3\ 170 = 1\ 960$	1 580	3 540
9..................	$\frac{100}{414} \times 3\ 170 = \ \ 765$	2 130	2 895

stresses are lowest in the columns that have the highest direct stresses. Thus, the total combined wind stress is: For Column 7, 3 805 lb per sq in.; for Column 8, 3 540 lb per sq in.; and for Column 9, 2 895 lb per sq in. All these unit stresses from wind are allowable increases, so that the column sections need not be increased for wind. Furthermore, the regularity of the design set-up permits that only a few calculations of an investigating character need be made since, as the analysis proceeds upward from the bottom of the building, all direct wind stresses in columns decrease uniformly to zero at the top, and the bending stresses increase slightly, because the section moduli decrease more rapidly than the column areas or wind bending moments. A few checks at intermediate levels between the top and the bottom of the building will reveal conditions to the designers. It is evident that in all this analysis the actual dimensions and the clear spans of the members are all important.

(*12*).—*Type and Placement of Bracing.*—The general wind-design procedure already outlined for one floor of Bent *B*, as shown in Table 4, should be followed throughout with all bents in each direction. For Bents *C* and *D*, full-story knee-braces could be used to advantage between elevators in the center panel, thereby effecting real economy in material and in field rivets. Knee-braces hidden in the exterior masonry in the center panel of Bent *A* would probably be an advantage also to take care of the heavy moments and shears. (See Fig. 4.)

In a longitudinal direction the building would be largely braced by four bents, marked *E* and *F*, which have a base width of 142 ft. The total wind load could be concentrated in these alone. In such event it would be a great economy to use full-story horizontal K-frames behind the elevators between Columns 16 to 29 and 15 to 30, as well as knee-braces in the court walls between Columns 14 to 31 and 17 to 28. By using proper sections the web systems will act as they have been pre-determined to perform, and general accuracy within reasonable limits will be obtained.

Using the dead load as the resisting moment for a uniformly distributed [illegible] wind load, [illegible] the dead load for a uniformly distributed wind load of [illegible] lb per sq ft. Therefore, the requirements of stability are fulfilled.

In Table [illegible] [illegible] the wind stresses in the columns [illegible]. It should be noted that the bending

TABLE [illegible]

[illegible]	[illegible]	[illegible]	Combined stress
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

stresses [illegible] the columns that have the highest direct stresses. Thus, the [illegible] stress in [illegible] is [illegible] lb per sq in.; for [illegible] lb per sq in. All these [illegible]. [illegible] the column [illegible] for wind. [illegible] the [illegible] of the [illegible] what [illegible] from the bottom of the building [illegible] wind stresses [illegible] uniformly to zero at the top, and [illegible] the section [illegible] the [illegible] moments. A few checks at intermediate levels between the top and the bottom of the building will reveal conditions to the designer. It is evident that in all this analysis the [illegible] of the members are all important.

([illegible]) *[illegible].*—The general wind design procedure already outlined for the [illegible] of Bent *B*, as shown in Table [illegible], should be followed throughout with all bents [illegible]. For Bents *C* and *D*, [illegible] knee-braces would be used to advantage [illegible] in the center panel, [illegible] saving in material and in field rivets. Knee-braces hidden in the [illegible] panel of Bent *A* would probably be an advantage to take care of the heavy moments and shears. (See Fig. [illegible].)

In a longitudinal direction the building would be [illegible] braced by four bents, marked [illegible] and [illegible], which have a base width of 142 ft. The total wind load could be concentrated in these alone. In such event it would be a great economy to use [illegible] behind the elevators between Columns 16 to [illegible] and [illegible] to 26, as well as knee-braces in the [illegible] walls between Columns 14 to [illegible] and 17 to 28. By using proper sections the web systems will act as they have been predetermined as working, and general economy within reasonable limits will be obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EVAPORATION FROM WATER SURFACES A SYMPOSIUM

Discussion

By Messrs. Ivan E. Houk, and R. I. Meeker

Ivan E. Houk,[40] M. Am. Soc. C. E. (by letter).[40a]—The papers by Messrs. Rohwer and Follansbee furnish a long needed digest of available evaporation data. Mr. Rohwer's paper brings together valuable information regarding comparative evaporation rates observed at different types of pans, and their relations to actual evaporation losses from large reservoir surfaces. Mr. Follansbee's paper presents comprehensive and valuable compilations of pan evaporation data reduced to equivalent reservoir evaporation.

As regards the ratios of pan evaporation to reservoir evaporation, it is not surprising that there should be some differences between results obtained at different locations. Considering the varying meteorological conditions throughout the country it would not be logical to assume that the ratio of the evaporation rate at any given pan to the evaporation loss from a large reservoir surface in the vicinity of the pan should have the same value in all localities. However, considerable porportions of the differences shown in Mr. Rohwer's tabulations are probably due to incidental errors of observations and lack of strictly comparable environment at the different places where the investigations were conducted.

The writer would not expect the data on pan ratios secured at the Stonyford, Calif., installations (Fig. 3), to be susceptible of general application, because of the proximity of the pans to each other and to the shore of the reservoir. The slightest offshore or onshore breeze must have produced essentially the same vapor pressure in the air immediately over the different pan surfaces; so that the only factor causing differences in evaporation rates at the different pans must have been the temperature of the water within the pans. Moreover, ratios of reservoir evaporation to pan evaporation at Stonyford must have been affected materially by the prevailing wind direction. In such an installation prevailing offshore breezes would mean definitely

Note.—This Symposium on Evaporation from Water Surfaces was published in February, 1933, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings* as follows: May, 1933, by Messrs. Ralph R. Randell, C. E. Grunsky, and Charles H. Lee; August, 1933, by Messrs. J. T. Harding, L. Standish Hall, and Adolpho Santos, Jr.; and November, 1933, by John W. Pritchett, Assoc. M. Am. Soc. C. E.

[40] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[40a] Received by the Secretary November 3, 1933.

lower ratios of reservoir evaporation to pan evaporation than would be the case if onshore breezes predominated.

In the Stonyford installation the writer would expect the evaporation rate at the floating pan to be essentially the same as at the sunken land pans. Somewhat different ratios of reservoir evaporation to pan evaporation undoubtedly would have been obtained at the different pans if the floating pan had been in the center of the lake and the land pans entirely away from the reservoir. In other words the grouping of the pans near the shore of the lake tended to eliminate one of the variable factors that affect the rate of evaporation. Consequently, engineers cannot apply, blindly, ratios determined at such installations to land pan measurements made in a desert country, entirely away from any large body of water. Such ratios can only be used in the case of pan measurements near large lakes or reservoirs, where the relative humidity above the pan is substantially the same as over the larger water areas.

Mr. Follansbee's paper supplies a wealth of information regarding estimated reservoir evaporation based on pan measurements in different sections of the United States. His tabulations can be used in preliminary investigations of proposed storage projects, selecting the stations most comparable as regards elevation, temperature, relative humidity, and wind velocity. The writer believes that Mr. Follansbee's calculated estimates of reservoir evaporation in Western United States are on the safe side in most cases; that is, that they tend to be too high rather than too low, if in error at all. The reason for this belief is that in a few cases the pan measurements may have been made in regions where the relative humidity is definitely lower than that at the locations where the pan ratios were determined, and also definitely lower than the relative humidity that will occur in the vicinity of the pan after a large reservoir is constructed.

The Yuma (Citrus) Station, in Arizona, (Table 8 (*r*)) is an example of the aforementioned conditions. In this case the writer believes that the coefficient of 0.69 used to reduce Class A pan evaporation to reservoir evaporation is entirely too high. If a large reservoir was built at the location of the Yuma (Citrus) Station the relative humidity of the air above the reservoir surface probably would not be materially different from the relative humidity of the air above a large reservoir at the Yuma (Date Orchard) Station. Consequently, since the wind velocities and air temperatures are practically the same at the two stations, the evaporation rates at the two reservoirs would be essentially the same and the ratio of reservoir evaporation to past pan records at the Yuma (Citrus) Station would be much lower than 0.69. Meteorological conditions at the Yuma (Citrus) and the Yuma (Date Orchard) Stations and their effects on measured pan evaporation were discussed in a previous paper by the writer.[41]

In finally planning any important storage development, the engineer should consider carefully the meteorological conditions at the proposed reservoir

[41] "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 282–285.

site before and after construction and their relations to the meteorological conditions at the places where the pan ratios were determined, before adopting a definite value for the ratio of reservoir evaporation to pan evaporation. The engineer might also, profitably, consult Mr. Rohwer's excellent and comprehensive *Bulletin*[42] which gives the results of recent investigations by the U. S. Department of Agriculture at Fort Collins, Nebr., Logan, Utah, Imperial and Lake Tahoe, California, and Fort Collins, Victor, and Pike's Peak, Colo. In that *Bulletin* Mr. Rohwer not only showed conclusively that the Dalton law, proposed in 1802, constitutes a satisfactory theoretical basis for a practicable evaporation formula, but also developed such a formula from his extensive evaporation measurements and tested it in many different locations and at many different altitudes throughout the country.

In order to use the Rohwer formula in a given case it is necessary to know the altitude, wind velocity, relative humidity, and water surface temperature. Since the altitude is practically always known with a sufficient degree of accuracy, only three factors remain to be determined. Of these three, wind velocity has already been measured more or less extensively at meteorological stations maintained by the U. S. Weather Bureau; so that the principal additional data needed are values of relative humidity and water surface temperature in different sections of the country, both of which quantities are easily measured with relatively inexpensive equipment. Engineers interested in securing better evaporation data should encourage the measurement of relative humidity and water surface temperatures at lakes and reservoirs whenever possible, as well as the establishment of additional evaporation stations and the measurement of all pertinent meteorological conditions at such installations.

R. I. Meeker[43], M. Am. Soc. C. E. (by letter)[43a]—Evaporation losses from reservoirs and river channels, in the Western United States, have become a matter of considerable importance to irrigation engineers, in recent years, due to interstate river problems and numerous proposed channel reservoirs of large capacity. Information and engineering data on evaporation from water surfaces were relatively meager before 1924.

In 1927, Ivan E. Houk, M. Am. Soc. C. E., brought together in compact form valuable information[44] now pertinent to engineering studies of river equalization. The Symposium on "Evaporation from Water Surfaces" is a fitting conclusion to the interest awakened in the profession by Mr. Houk's paper.

The contributions of Mr. Rohwer and of Mr. Follansbee represent extensive research, exacting compilation, and careful analytical study of an immense mass of records—State, regional, continental, and foreign. Table 8 in Mr. Follansbee's paper is a most valuable reference work because of its

[42] "Evaporation from Free Water Surfaces," by Carl Rohwer, Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 271*, U. S. Dept. of Agriculture, Washington D. C., December, 1931.

[43] Cons. Engr., Denver, Colo.

[43a] Received by the Secretary November 25, 1933.

[44] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 266.

comprehensive character. Table 10 is most instructive, and forces one to study comparative values of evaporation from the factors presented.

The conclusion of the Sub-Committee, in favor of the Class A land pan as to desirability for evaporation records is sustained by the experience of the writer. No floating-pan records secured by him were ever satisfactory as to accuracy, due to indeterminate water gains or losses from wave action.

These papers on evaporation, supplemented by "Evaporation from Free Water Surfaces"[45], by Mr. Rohwer, will furnish a satisfactory library for years to come on that important subject.

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[45] *Technical Bulletin No. 271,* U. S. Dept. of Agriculture, Washington, D. C.

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DISCUSSIONS

DEVELOPMENTS IN REINFORCED BRICK MASONRY

Discussion

By James H. Hansen, Jun. Am. Soc. C. E.

James H. Hansen,[32] Jun. Am. Soc. C. E. (by letter)[32a].—It has been suggested by Messrs. Hatt and McBurney that the writer should have given, in more detail, the qualifications pertaining to the various recommended values for brick masonry and brick strengths. Their criticisms are legitimate and should be taken into consideration by any one whose purpose it is to develop the primary assumptive values to be used in the design of reinforced brick masonry. The object of the paper, however, was to give a short, intelligible résumé of what had been done. Obviously, the variations in all types of brick and brick masonry through geographic location or workmanship could not be discussed without involving a mass of detail from which it would be difficult to derive conclusions. Furthermore, the qualifications pertaining to the recommended allowable working stress of 600 lb per sq in. for brick masonry, which Professor Hatt states should be given, are of such a nature as to be vague and almost indeterminable. For instance, what constitutes a "thoroughly inspected joint," and when are the "effects of eccentric and concentrated loads and lateral forces" fully considered? Such qualifications are commendable cautions, and serve to indicate that brick masonry of such a nature can be (and often is) obtained, and that, therefore, it should be given legitimate consideration in the development of reinforced brick masonry.

Tests conducted by Inge M. Lyse, Assoc. M. Am. Soc. C. E., under the sponsorship of Mr. Judson Vogdes substantiate that opinion. Mr. Lyse reports tests[33] on fifteen columns, in which average solid brick strengths were 13 760 lb per sq in. flat, 10 680 lb per sq in. on edge, and 10 070 lb per sq in. on end. The columns were $12\frac{1}{2}$ in. square and 10 ft high. The maximum

Note.—The paper by James H. Hansen, Jun. Am. Soc. C. E., was published in March, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1933, by Messrs. W. K. Hatt, and C. T. Schwarze; and October, 1933 by Messrs. J. W. McBurney, and E. G. Walker.

[32] Acting Mgr., New York Brick Mfrs. Assoc., New York, N. Y.

[32a] Received by the Secretary November 8, 1933.

[33] *Engineering News-Record*, March 16, 1933.

loading with cement-mortar and no reinforcement in one case was 800 000 lb, or about 5 120 lb per sq in. Furthermore, Mr. Hugo Filippi reports the results of a large testing program[34] in which plain brick piers, approximately 8 by 8 by 17 in., took average maximum loads of 2 296 lb per sq in.

Professor Schwarze has suggested that plastic flow in brick masonry is less than in concrete. No long-time load tests have been conducted, to the writer's knowledge, to establish this contention. However, it does seem probable that such is the case and that the distribution of stresses based on the values of *n*, as customarily established, will result in more accurate stress determinations in brick masonry than in concrete. This may prove a decided advantage for this type of construction. In correspondence with the writer, Professor Schwarze states that recent tests under his supervision have strengthened his belief on this point. These tests on six piers, 8 in. square by 30 in. high, reinforced with $\frac{1}{2}$-in. round rods and $\frac{1}{8}$-in. ties, 6 in. on center, revealed no evidence of plastic flow and gave results of 1 500 lb per sq in. in compression when rusted rods were used. Failure was caused by crushing of the brick, and there was no indication of bond failure. The value of *n* for column design was established as 12. Other tests have recently been completed, and the writer will be glad to furnish any one interested with the detailed reports.

[34] *Brick Engineering*, Vol. III.

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DISCUSSIONS

ACTUAL DEFLECTIONS AND TEMPERATURES IN A TRIAL LOAD ARCH DAM

Discussion

BY MESSRS. IVAN E. HOUK, AND A. V. KARPOV AND R. L. TEMPLIN

IVAN E. HOUK,[20] M. AM. SOC. C. E. (by letter).[20a]—The experimental measurements at Ariel Dam show three facts: First, that internal temperature problems are important in the design and construction of concrete dams; second, that effects of internal temperature changes can be controlled adequately by proper precautions in planning and building large concrete structures; and, third, that the amplified trial-load method of analyzing arched masonry dams, including proper allowances for effects of twist, tangential shear, and foundation and abutment deformations, furnishes a satisfactory basis for the design of such structures.

Temperature changes in the interior of concrete structures have been measured and reported in engineering literature many times during the last thirty years. As far as the writer can determine, the investigations made by Thaddeus Merriman, M. Am. Soc. C. E., at the Boonton Dam, in New Jersey, during the period from 1903 to 1906, constituted the first real attempt to measure and analyze temperature changes in the interior of a large masonry dam.[21] This was mentioned in a résumé of all available concrete temperature observations prepared by the writer in 1930 and 1931. In two papers[22] the writer included tabulations of maximum concrete temperatures, maximum concrete temperature rise caused by generation of chemical heat during the setting period, maximum range in concrete temperature caused by seasonal variations in air and water temperatures at the faces of the structures, and pertinent descriptive data, with discussions of certain concrete temperature investigations being conducted by the U. S. Bureau of Reclamation at that time.

NOTE.—This paper by A. T. Larned and W. S. Merrill, Members, Am. Soc. C. E., was published in May, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1933, by George Jacob Davis, Jr., M. Am. Soc. C. E.; October, 1933, by Messrs. D. C. Henny, and B. E. Torpen; and November, 1933, by Messrs. Lars R. Jorgensen and H. B. Muckleston.

[20] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[20a] Received by the Secretary October 28, 1933.

[21] *Transactions*, Am. Soc. C. E., Vol. LXI (December, 1908), pp. 413–429.

[22] "Temperature Variations in Concrete Dams," by Ivan E. Houk, *Western Construction News*, December 10, 1930, pp. 601–608; also, "Setting Heat and Concrete Temperatures," by Ivan E. Houk, *Western Construction News*, August 10, 1931, pp. 411–415.

Various additional data on concrete temperatures have been published in foreign engineering periodicals, and in the Final Report of the Engineering Foundation Arch Dam Committee,[23] since the preparation of the aforementioned tabulations. However, the paper on the Ariel Dam investigations furnishes the first comprehensive treatment of the subject in all its varied relations to the design and construction of a large arch dam.

The authors gave a value of 143.4° F for the maximum concrete temperature observed at Ariel Dam, and a value of 74° F for the maximum placing temperature; but they do not state whether these measurements apply to the same location in the dam, or whether they can be used in calculating the maximum rise in concrete temperature caused by generation of chemical heat during the curing period. The writer would like to know what the maximum temperature rise was in any part of the structure and where, and under what conditions, it occurred?

Considering the dimensions and comparatively rapid rate of construction, it is probable that the resistance thermometers placed near the center of Ariel Dam were in a condition of practically perfect insulation from the time of installation until the chemical actions accompanying the curing of the concrete were completed. Consequently, the maximum rise in concrete temperature shown by these thermometers would be a measure of the maximum rise to be expected in a concrete structure of any size, assuming it to be built with the same brand of cement, the same kind of aggregate, and the same proportions of concrete ingredients used at Ariel.

In preparing the aforementioned résumé of available concrete temperature data the writer found that the maximum rise measured in a large masonry dam up to that time, reduced to a common basis of 1 bbl of cement to 1 cu yd of masonry, occurred at the Bull Run Dam, near Portland, Ore., and amounted to 63.4° F. Incidentally, it is understood that laboratory tests of concrete specimens from the Bull Run and Ariel Dams showed very similar thermal properties. However, the temperature data for Owyhee Dam, have been revised[24] on the basis of later concrete temperature measurements and show somewhat higher values. The maximum temperature of 120° F, and the maximum temperature rise of 50° F, given in the original compilation should be increased to 125.3° F, and 65.9° F, respectively; so that the maximum rise observed in Owyhee Dam was slightly higher than that at Bull Run Dam.

Considerably higher maximum temperatures and maximum temperature rises due to setting heat have been observed in structures built with richer concrete mixtures, as, for instance, in tunnel linings and large building foundations. A maximum rise of 100° F was observed in the construction of the strong-room of the Bank of England a few years ago.[25]

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[23] Engineering Foundation Arch Dam Investigation, Rept. by Committee, Vol. III, The Engineering Foundation, May, 1933.

[24] *Western Construction News*, August 10, 1931, Tables 1 and 2, pp. 413–414.

[25] "Temperature and Humidity Effect on Concrete Defined." *Concrete*, June, 1929, p. 52; abstract of a Lecture before the Royal Inst. of British Archts., in London, England, by Oscar Faber.

In studying the variations in lake temperature, shown on Figs. 18 and 20, it would be desirable to know what changes in reservoir surface elevation occurred during the periods of time represented by the curves, since seasonal variations in water temperature are dependent, to a large extent, on the depths of water above the elevations of temperature measurement. Measurements of the temperature of reservoir water at different depths are important because of its direct effect on the temperature of the concrete near the up-stream face of the dam.

The writer would like to know what kind of resistance thermometers were installed in Ariel Dam? what kind of cables were used in extending the electrical connections to the switchboards? and whether the leads within the masonry were placed in metal conduits? Inadequate design of thermometers, thermometer lead connections, and thermometer lead insulation constituted one of the principal sources of trouble in connection with early investigations of concrete temperature variations. It was not at all unusual for from 25 to 50% of the total number of thermometers in a single structure to become defective in some of the earlier investigations due to such causes, long before a satisfactory series of temperature measurements was secured.

Unfortunately, resistance thermometers made by the manufacturers of scientific instruments, such as were used in many of the early measurements of concrete temperature, were designed primarily with the idea of providing a high degree of precision in the observations rather than continuous satisfactory service through long periods of time under the conditions existing in such structures. Resistance thermometers installed in the interior of a masonry dam need not be read closer than the nearest degree. However, they do need to be capable of indicating actual concrete temperatures within 1° of the true values continuously during a period of time lasting several years after the completion of the dam.

The resistance thermometer used in the concrete temperature investigations at Gibson Dam, a 200-ft concrete arch dam constructed on the Sun River Project, in Montana,[26] a few years ago, was a substantial, home-made instrument, consisting of an insulated resistance coil placed in an 8-in. sheath of standard $1\frac{1}{4}$-in. galvanized pipe. The pipe was capped at both ends, and a hole provided in the center of one cap to serve as an exit for the armored cable leads. The resistance coil was held firmly in place inside the sheath, and adequately water-proofed, by pouring the pipe full of pothead compound through a small hole near the exit of the leads. The instrument was placed in freshly deposited concrete, and the leads laid in a shallow trench excavated at the surface of the lift before the concrete hardened, no metal conduit protection being provided.

Similar, but slightly modified, home-made thermometers were installed in Owyhee Dam, a 421-ft, curved, gravity concrete dam recently completed on the Owyhee Project, in Eastern Oregon. The modification consisted of lengthening the resistance coil so as to reduce the heating of the instrument during readings, providing No. 16 A. W. G. rubber insulated, fixture wire leads,

[26] *Western Construction News*, December 10, 1930, p. 604.

12 in. long, and placing a tee-and-nipple splicing chamber at the end of the thermometer sheath, so that the 12-in. leads could be properly connected to the main cable lengths at the time of installation. Lead-covered, armored cable leads were used on 113 of the 127 thermometers installed for general temperature observations, and special types of rubber-insulated cable on the remaining 14, no metal conduits being used for any of the leads. Three instead of two conductor cables were used, so that the resistance of the leads could be measured and deducted from the total resistance of the thermometer circuit each time a reading is made.

Thus far, only 3 of the 31 thermometers installed in Gibson Dam, and only 6 of the 113 armored, cable-connected instruments installed at Owyhee Dam have become defective. The thermometers at the Gibson Dam have been in place a little more than five years, and those at the Owyhee Dam about two years.

Some engineers have objected to the use of lead-covered, armored cable for leads on resistance thermometers embedded in concrete, because of the possibility of damage during installation and the tendency of the calcium hydroxide in the green concrete to corrode the lead covering during the early part of the curing period. One engineer predicted that all the Gibson Dam thermometers would be out of order within a year, due to the corrosive action alone. However, the record of the Gibson Dam shows that the detrimental effects of the calcium hydroxide on the lead sheath are of minor importance as compared with the advantage of securing absolute water-tightness during the early stages of installation. Apparently, the lead covering furnishes an additional protection against damage of insulation by workmen, or by moisture penetration, at the time such protection is most needed. The different types of improved rubber insulated cables being tried in the Owyhee Dam installations have not been in place long enough to furnish accurate comparative data. Rubber insulated cables, placed in well-drained, metal conduits, may prove adequate for long-time measurements. Nevertheless, from the standpoint of securing satisfactory data on concrete temperatures through a period of several years, the writer prefers rubber insulated, lead-covered, armored cable leads, suitably protected at contraction joint crossings.

The writer would like to know if any cores were obtained in blocks artificially cooled; and, if so, how the strength of such cores compared with the strength of those drilled in other sections of the dam? He would also like to know if any strength tests were made on the artificially cooled experimental block mentioned in Item 1 under the heading, "General Observations," and what average results were obtained if such tests were made? In other words, did the increased rate of heat dissipation due to artificial cooling cause beneficial or detrimental effects on the strength of the finished product; and, if so, what were the nature and extent of such effects? Obviously, the resultant effects on the strength and safety of the structure are the most important matters to be considered in determining whether or not refrigeration of concrete should be resorted to in building a large masonry dam. For

instance, such effects are much more important than the advantages or disadvantages of the method from the construction standpoint.

As regards the comparison of calculated and measured deflections in the arch section of the dam, immediately following Fig. 22, it might be pointed out that an additional reason for the difference in deflections at the center of the arch is the absence of uplift pressure in the cantilever elements noted in Item 6, under "General Observations." The design assumption of uplift pressure over one-half the horizontal area of the base, together with the assumption of cracking in the tension areas of the vertical sections, as mentioned under "Design of Arch Dam," would naturally result in an excess of calculated deflections over measured deflections. This would mean that the cantilever elements near the center of the dam are carrying more load, and the arch elements less load, than was calculated in the trial-load analysis. It would also mean that the actual maximum stresses in the loaded dam, in both arch and cantilever elements, are somewhat smaller than those determined by the trial-load analysis in the latter part of "Design of Arch Dam."

The fact that the measured abutment movements at Ariel Dam are somewhat greater than the deflections calculated on the basis of the theory of elasticity, may be partly due to the opening of cracks, seams, or fissures in the natural rock formations. The writer has never been satisfied that natural canyon walls can be considered as homogeneous elastic bodies and that their movements can be determined accurately by a mathematical treatment of the various resultant forces.

The deflection measurements at Ariel Dam, considered as a whole, constitute a satisfactory check on the design methods. There are always many uncertainties involved in attempting to represent actual field conditions at a large masonry structure by definite mathematical formulas. Nevertheless, the Ariel measurements show that such representations can be made with a reasonable degree of confidence. They confirm the conclusion previously reached by the writer as a result of his work on models of arch dams and on technical studies connected with the design of Boulder Dam—namely, that the amplified trial-load method of analysis, making proper allowances for twist, tangential shear, foundation deformations, and other important effects—can be used safely as a basis for the design of important arch or curved gravity dams.

A. V. Karpov[27] and R. L. Templin,[28] Members, Am. Soc. C. E. (by letter)[28a].—This paper is of great value and is indicative of the modern trend to base the design of dams not on arbitrary theoretical assumptions, but on data that have been checked and proved by actual tests. As far as the temperature measurements are concerned, the paper discloses a thorough experimental study. However, the deflection measurements were made with less thoroughness and, unfortunately, strain or stress measurements were omitted.

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[27] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[28] Chf. Engr. of Tests, Aluminum Co. of America, New Kensington, Pa.

[28a] Received by the Secretary November 3, 1933.

For a comprehensive study of the behavior of a dam, the temperature measurements as well as the deflection measurements are important and necessary, but they can hardly be considered as the final step in an investigation of this kind.

From the modern engineering point of view, the determination of stresses is the final goal of any experimental structural testing program, and all measurements are of value only as intermediate steps leading to that final goal.

Deflection Measurements.—There does not seem to be a generally accepted method of deflection measurements. Three methods were used in the past: (1) In 1926 the clinometer was used to measure the deflection of the Stevenson Creek Dam[29]; (2) between 1921 and 1928 the deflections of a number of dams in Switzerland were determined by precise triangulation[30]; and (3) in 1930 and 1931 the deflections of the Calderwood Dam were determined by direct measurements, using a special tape, and measuring the change of distances from a central pier.[31]

Every one of these methods, if properly applied, gave satisfactory results as far as the relatively large radial deflections are concerned. None of them can be considered as giving satisfactory results if the small tangential deflections are to be determined exactly. The first and third methods require that the down-stream face of the dam be made accessible during the deflection measurements. The second method can be applied without having access to the down-stream face of the dam during the tests.

The writers studied the possibility of applying all three methods, and came to the conclusion that the triangulation method has advantages when only a few measuring stations are to be used. Measuring the deflections at a large number of stations requires considerable work to evaluate the data and, consequently, the results of the tests are available only after an appreciable time has elapsed. This method has one distinctive advantage; it permits an easy study of the foundation and ground movements.

The clinometer has considerable advantages if measurements are made on a large number of stations; but foundation and ground movements can scarcely be studied by this method, unless some additional provisions are made.

Direct measurements with a tape and by the use of a central pier represent a reasonable method which makes it possible to measure deflections at a comparatively large number of stations; and, at the same time, a study of the ground movements is possible, particularly if more than one pier is used.

There seems to be a tendency to prefer the triangulation method, but the writers believe that, in many instances, this preference is based rather on an insufficient appreciation of the other two methods, than on the advantages of the triangulation method.

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[29] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

[30] "Deformationsmessungen an Staumauern," von W. Lang, Verlag der Abteilung für Landestopographie Bern, 1929.

[31] See p. 1565.

Deflection Measurements Made in the Ariel Dam.—One of the most recent and very important advances in the arch dam theory is the realization that such a dam cannot be considered independently, but that it is an integral part of the surrounding canyon. Consequently, it is impossible to draw a sharp division line between the behavior of the structure proper and the ground that supports it.

In making an experimental study of a dam the behavior of the ground is as important as that of the dam proper and it is hardly possible to arrive at satisfactory conclusions unless the behavior of the ground is disclosed. It seems to the writers that in the measurements at Ariel Dam the ground conditions were neglected somewhat; at least, the published data and diagrams do not give sufficient information about the deflections of the ground.

As far as the authors' conclusions, based on deflection measurements, are concerned, the writers cannot concur with a number of statements: (1) The authors state that the agreement reached on the Ariel Dam can be characterized as "entirely satisfactory" (Conclusion 8). Not only is there a considerable difference in the values of deflections, but even the character of the computed and measured deflection curves is different; (2) in Item 3 under "Deflections of the Arch Dam" the authors state that "from the maximum point the deflections taper off smoothly to the abutments and foundations." The July, 1932, measurements not only do not support, but rather contradict, that statement. The deflections at Elevations 120 and 180, as well as at the vertical section on the *E* line, clearly show this contradiction; (3) in Item 7, under "Deflections of the Arch Dam," the authors make a statement that the arch action took place as expected, basing their conclusion on the close agreement between measured and calculated tangential displacements. It is difficult for the writers to agree, considering the large discrepancies between the measured and computed radial deflections. The tangential displacements are small, and these measurements are, of course, much less reliable than those of the radial deflections. Considering both the radial deflections and tangential displacements, it would seem that the arch action was quite different than was expected.

Considering the radial deflections, it would appear that, as far as can be judged from Fig. 22, the agreement, in a majority of cases, between the measured and computed deflections, when adjusted only radially, is closer than between the measured and computed deflections when adjusted finally. This would seem to indicate that either the design assumptions or design method, or both, are incorrect. Furthermore, it would seem that the disagreement between computed and measured deflections would increase if the design was changed so as to take care of the actual deflections (which were larger than originally assumed) of the concrete thrust-block and of the south abutment (see Items (5) and (6), under "Deflections of the Arch Dam").

Determination of Stresses.—The paper does not disclose the magnitude of the actual stresses in the Ariel Dam, but the reader is left under the general impression that since there seems to be a fairly close agree-

ment between the computed deflections and those actually measured, an agreement between the computed and actual stress should be expected. This is a surmise which is not proved either in the Ariel Dam, or by any other commercial dam. Until that gap is bridged, there can be no assurance that a design is safe, rational, and economical.

In general, it is much easier to arrive at a close agreement between the computed and actual deflections than between computed and actual stresses. Examining the measurements described in the paper from this point of view, the writers believe that the results of the deflection measurements rather indicate that there is a very considerable difference between the computed and actual stresses in Ariel Dam. While the tests described by the authors are quite in conformity with the published statement of the Engineering Foundation Committee on Arch Dam Investigation,[32] yet the results obtained by use of different methods in the tests conducted by the writers would appear to substantiate the comments made in this discussion.

Trial-Load Method.—The Ariel Dam was designed painstakingly, using the trial-load method, and according to the best standards accepted in connection with this method. Realizing the fact that the customary assumption of linear distribution of stress is far from being correct, an additional refinement was attempted. In order to compensate for the non-linear distribution of stress, the very doubtful mean of assuming an entirely arbitrary high modulus of elasticity in shear was used, as seems to be customary in the latest designs made by the trial-load method.

The tests described in the paper are of great interest and show that although the trial-load method may be an advance over design methods of the past, nevertheless, it apparently still admits of considerable improvement. Probably the most important improvement lies in the shape of the horizontal arches.

At present, it is generally admitted that notwithstanding the fact that the hydrostatic water pressure is constant at every horizontal elevation, the load imposed on a horizontal arch varies, a division of the total load between the arch and the vertical elements taking part. In spite of that knowledge, the arch dam is one of the few modern structures subjected to hydrostatic pressure in which the shape of the structure is arbitrarily chosen. The circular shape of the horizontal arches is about the same as if an attempt should be made to build non-circular large-sized tubing to be subjected to a uniform hydrostatic pressure; this would result not only in poor utilization of material, but the exact evaluation of stresses in such a tube would be of extreme complexity. A dam composed of circular horizontal arches is quite similar, but instead of a non-circular tube subjected to uniform hydrostatic load, a circular arch is subjected to non-uniform load.

Next, it is necessary to recognize the fact that a dam cannot be built as a monolithic structure, and, consequently, a design based on the assumption of such a structure cannot represent the actual conditions. The trial-load

[32] "Arch Dam Investigation," p. 6, Vol. III May, 1933, pub. by Engineering Foundation.

method simply neglects the existence of the vertical joints in spite of the fact that such joints influence the stress distribution considerably. Such a method results in considerable difference between the actual and the design stresses. One advance in arch dam design (that is excluded by the present trial-load method) will be made possible by the clear recognition of the fact that if the dam is built by current methods, the vertical joints are inevitable, but may be arranged so as to be useful.[33]

Finally, the close correlation between the behavior of the ground and of the dam proper is to be taken into consideration. This correlation is partly expressed in the advantages of avoiding sharp changes in conditions at places where the dam proper meets the foundations or abutments. The consequent liberal use of properly designed fillets is again an important design improvement that would be made clear if, instead of the linear distribution of stress and high modulus of elasticity in shear, some more scientific assumptions were introduced.

The usual trial-load method requires considerable time and effort, but neglects entirely, or partly, these important considerations. Arch dams are very responsible and expensive structures, and the expenditure of time and effort in their design is fully justified, provided the evaluated stresses are reasonably close to those obtained in the actual structure, assuring the safety and economy of the design. As the measurements on Ariel Dam clearly show, in spite of the painstaking design, a considerable difference appears between the actual conditions and the conditions computed by the use of this method.

The increase of engineering knowledge in design of dams in general and arch dams in particular is of extreme importance. The data presented in the paper show that the engineering advance made in the past few years is not sufficient. Much more theoretical and research work is necessary to enable a dam to be designed on the basis of sound engineering knowledge, as is imperative for structures of that size and responsibility.

The foregoing criticisms do not detract in any way from the value of the paper, and are made with the idea of indicating that the design of arch dams has reached a point when further advance depends on measurements of a more complicated nature than were attempted in Ariel Dam.

[33] See "The Compensated Arch Dam," by A. V. Karpov, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 1309.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EARTHS AND FOUNDATIONS
PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY N. J. DURANT, ASSOC. M. AM. SOC. C. E.

N. J. DURANT,[74] ASSOC. M. AM. SOC. C. E. (by letter).[74a]—This report forms a valuable addition to the existing, but scarce, knowledge of foundation pressure distribution. Its value is enhanced by the inclusion of data obtained from actual structures which, in conjunction with the experimental values, are shown to be particular cases of a comprehensive theory given in the report.

This is the true method of research: The formation of the theory first, modified afterward by the inclusion of only those variables which experiment shows to be of significance. Without the theory investigators are left with a set of isolated results from which no generalizations are possible. The most that could result would be empirical formulas, with which the science of engineering is already overburdened.

The usual assumption made for foundation pressures is that, due to vertical loads, the pressure distribution is uniform. No procedure in design, however, can be justified on the grounds of simplicity if the true state of affairs can be made manifest by simple analyses, unless it provides adequately for all stresses to which the system is subjected.

There are minor faults in the report which may be a source of trouble to those who would wish to understand how the results are obtained.

For instance, Equation (1) should be written, $p_z = \frac{1}{2_n} \frac{n!}{\left(\frac{n}{2}\right)^2!}$; otherwise, the results given in Fig. 1 will not be obtained. It will be noticed also that

NOTE.—The Progress Report of the Special Committee on Earths and Foundations was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in May, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: August, 1933, by Messrs. L. C. Wilcoxen, H. de B. Parsons, William P. Kimball, and T. A. Middlebrooks; September, 1933, by Messrs. Daniel E. Moran, and A. E. Cummings; October, 1933, by Messrs. Edwin J. Beugler, Jacob Feld, George D. Camp, and Charles Terzaghi; and November, 1933, by Messrs. John H. Griffith, Harry E. Sawtell, and O. K. Froehlich.

[74] Designer, Bridge Dept., Rendel, Palmer & Tritton, London, England.

[74a] Received by the Secretary September 30, 1933.

although n may have any integral value, p_z has a meaning only when n is an even integer. The difficulty is easily overcome by the application of the gamma function for $\Gamma(n+1) = \sqrt{2\pi n}\, n^n\, e^{-n}$ (when n is large). This is Stirling's formula. Equation (2) which is correct, then follows.

Again, the solution of the differential equation (Equation (18)), for the hydro-dynamical excess, has the appearance of artificiality and gives the idea that it has been solved by tentative methods. All such equations are capable of direct solution in the functional or particular form, the latter form being, from the physicist's or engineer's viewpoint, the more convenient.

The functional form necessarily leads to the particular form when boundary conditions are present. The processes of solution in the two cases, however, are fundamentally different. The direct solution in the particular form is obtained as follows: For the conditions, $t > 0$ and $0 < z < 2\,h_0$, Equation (18), which gives the hydro-dynamical excess in a previous medium (evaporation being absent), is derived by means of the two-dimensional form of Green's theorem. Accepting the equation, and using the notation given in the report, the boundary conditions are: $w = w_o$ when $z = 0$; $w = w_1$ when $z = 2\,h_0$; and, $w = f(z)$ when $t = 0$. To simplify the boundary conditions, the dependent variable, w, is changed *to* ϕ. Thus:

$$w = w_o + (w_1 - w_o)\,\frac{z}{2\,h_o} + \phi \;\ldots\ldots\ldots\ldots\ldots\ldots(65)$$

Then, if $t > 0$ and $0 < z < 2\,h_0$, ϕ satisfies the equation:

$$\frac{\partial \phi}{\partial t} = c\,\frac{\partial^2 \phi}{\partial z^2} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(66)$$

With $\phi = 0$ when $z = 0$ and when $z = 2\,h_0$; and with $\phi = f(z) - w_0 - (w_1 - w_0)\,\dfrac{z}{2\,h_0}$, when $t = 0$:

$$\phi = T\,Z \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(67)$$

in which, T is a function of t and Z is a function of z. Then Equation (66) becomes:

$$\left(\frac{1}{c}\right)\left(\frac{1}{T}\right)\frac{dT}{dt} = \left(\frac{1}{Z}\right)\frac{d^2 Z}{dz^2} \;\ldots\ldots\ldots\ldots\ldots\ldots(68)$$

Since each side of the equation is independent of the independent variable on the other, each side must be a constant, say, K. Therefore,

$$\phi = e^{cKt}\left\{A\,e^{z\sqrt{K}} + B\,e^{-z\sqrt{K}}\right\}$$

and, $\phi = 0$ when $z = 0$, or $2\,h_0$:

$$\phi = A\,e^{cKt}\left\{e^{z\sqrt{K}} - e^{-z\sqrt{K}}\right\} \;\ldots\ldots\ldots\ldots\ldots\ldots(69)$$

In order that ϕ may be zero <u>when</u> $z = 2\,h_0$ and still remain finite when t approaches ∞, the radical, $\sqrt{K}$ must be imaginary. Then, $2\,h_0\,\sqrt{K}$

must be of the form, $i\, m\, \pi$, in which, m is any integer. Therefore,

$$\phi = \sum_{m=1}^{\infty} c_n\, e^{-f} \sin \frac{m\, \pi\, z}{2\, h_o} \quad \text{.................... (70)}$$

in which, $f = \dfrac{c\, m^2\, \pi^2}{4\, h^2_o}\, t$, and c_n is the Fourier coefficient; that is,

$$c_n = \frac{1}{h_o} \int_0^{2\, h_o} \left\{ f\,(z) - w_o - (w_1 - w_o)\, \frac{z}{2\, h_o} \right\} \sin \frac{m\, \pi\, z}{2\, h_o}\, d\,z$$

or,

$$c_n = \frac{1}{h_o} \int_0^{2\, h_o} f\,(z) \sin \frac{m\, \pi\, z}{2\, h_o}\, d\,z + \frac{2}{m\, \pi} \left\{ w_1\,(-1)^m - w_o \right\} \quad \text{.... (71)}$$

therefore,

$$w = w_o + (w_1 - w_o)\, \frac{z}{2\, h_o} + \frac{2}{\pi} \sum_{m=1}^{\infty} \frac{w_1\,(-1)^m - w_o}{m}\, e^{-f} \sin \frac{m\, \pi\, z}{2\, h_o}$$

$$+ \frac{1}{h_o} \sum_{m=1}^{\infty} e^{-f} \sin \frac{m\, \pi\, z}{2\, h_o} \int_0^{2\, h_o} f\,(z) \sin \frac{m\, \pi\, z}{2\, h_o}\, d\,z \quad \text{.......... (72)}$$

If $w_o = w_1 = 0$, then,

$$w = \frac{1}{h_o} \sum_{m=1}^{\infty} e^{-f} \sin \frac{m\, \pi\, z}{2\, h_o} \int_0^{2\, h_o} f\,(z) \sin \frac{m\, \pi\, z}{2\, h_o}\, d\,z \quad \text{.......... (73)}$$

It will be seen that Equation (72) differs in an essential respect from Equation (23) of the report, but Equations (24) and (73) are identical.

The photo-elastic method substantiates Boussinesq's curves of pressures, thus providing a confirmation of experiment and theory. This experimental method in which use is made of polarized light is of extreme value in cases in which analysis alone would be difficult, tedious, or impossible. As the stress function is independent of the elastic constants any transparent material free from initial stress may be used.

The method is based on the fact (established by experiment) that an isotropic transparent material becomes doubly refracting when stressed, the principal axes of stress at a point coinciding in direction with the optical principal axes. In addition to the work of Professor Coker, cited by the Committee,[5] H. Neuber, of Munich, Germany, has published a grapho-analytical method[75] by which the complete field of stress may be obtained. This method is short, simple, and accurate.

The members of the Committee are to be congratulated on their findings and the Society for publishing in a convenient form so much valuable information.

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[5] "A Treatise on Photo-Elasticity," by L. N. G. Filon and E. G. Coker, Cambridge Univ. Press.

[75] "New Method of Deriving Stresses Graphically from Photo-Elastic Observations," *Proceedings*, Royal Soc., August, 1933.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

WATER-POWER DEVELOPMENT OF THE ST. LAWRENCE RIVER

Discussion

BY MESSRS. ADOLPH J. ACKERMAN, AND L. F. HARZA

ADOLPH J. ACKERMAN,[37] ASSOC. M. AM. SOC. C. E. (by letter)[37a].—The early history of the St. Lawrence Waterway project seems to be developing with surprising similarity to the early events in the planning of the Panama Canal. The general features of both projects quite obviously run along parallel lines: They are the two most important artificial navigation routes in the Western hemisphere; the estimated cost to complete the Canal when the United States took possession was about the same as the present estimated cost of the Waterway; engineering and construction problems are of similar magnitude and beyond ordinary conception; matters of policy in each project are of international importance; both are of vital concern to the same industry, the railroads, and where the Waterway at present has its chief controversy in the single-stage *versus* the two-stage plan, the Canal was debated in highest circles on the feature of the sea-level type *versus* the lock type. It is a matter of record that a majority of the Board created in 1905 to study the problem reported in favor of the sea-level canal. Subsequently, however, the Senate, giving heed to the Minority Report, passed a bill providing for the lock canal.

The present-day visitor to Panama, who heretofore has more or less taken the Canal "for granted," is at once impressed by the profound logic of its general plan. Such approval, in the end, is the supreme mark of engineering achievement and of unselfish administration of a public trust. It is, obviously, the basis on which Professor Mead has founded his analyses in calling attention to apparent deficiencies in the plans now being advocated in the Treaty negotiations for the Waterway.

NOTE.—The paper by Daniel W. Mead, Hon. M. Am. Soc. C. E., was presented at the Joint Meeting of the Power Division of the Society with the American Institute of Electrical Engineers and the Hydraulics Division of the American Society of Mechanical Engineers, Chicago, Ill., June 29, 1933, and was published in August, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: August, 1933, T. H. Hogg, M. Am. Soc. C. E.; and November, 1933, by Messrs. Theron M. Ripley, W. S. Lee, Frank E. Bonner, Rufus W. Putnam, James W. Rickey, Walter M. Smith, J. W. Beardsley, and L. F. Harza.

[37] Constr. Plant Designing Engr., Tennessee Val. Authority, Knoxville, Tenn.

[37a] Received by the Secretary October 16, 1933.

At this time it seems likely that the parallelism of the St. Lawrence Waterway and the Panama Canal may be continued. It remains to be seen to what extent the "minority opinion" will receive adequate consideration or will influence future acts.

An unusual opportunity exists on the St. Lawrence Waterway project to use an engineering model to a greater extent than heretofore has ever been attempted. The importance of preliminary investigations by means of models has received general recognition in the United States only in the past few years. One of the greatest current examples is found in the Mississippi flood-control problem where engineering policies and traditions which have stood inviolate for fifty years or more, are boldly being replaced by a new policy of scientific planning on hydraulic models. It has been ably stated[38] that all these changes of policy have long had advocates. They have come to fruition now because of engineering direction that does not fear to experiment. Boldness of engineering initiative buttressed by experimental research has been the inspiration of the new planning. This spirit of research and of willingness to put its conclusions to actual trial deserves express recognition. It has prevailed in every rank of the Engineering Profession.

It is the responsibility of the Engineering Profession to earn this sort of recognition in planning the St. Lawrence Waterway. There are numerous problems of operating conditions and their effect on navigation, ice movements, erosion, river control, and construction procedure and hazards, which may be clearly understood by the few engineers who have devoted the greater part of their lives to a study of the St. Lawrence River, but it is important that their opinions and deductions be studied in the most scientific manner, and compared with any new findings, before becoming rigid policies.

In the past decade engineers have adopted the science of models to a point where practically all important water-power projects are designed after observing the hydraulic performance of the principal elements on models. However, very often the general features had become established sufficiently to prevent the adoption of major improvements indicated by the model. It is safe to predict that hydraulic models of the St. Lawrence project will eventually be built. By building a comprehensive model as the first step, on which the various proposed plans may be studied in sufficient detail to demonstrate all conditions of operation, navigation, and construction procedure, the project is assured of the soundest basis of planning, and the best answer is given to those clamoring for immediate action.

Not only will disagreement among engineers be dispelled and individual definitions of the term, "hazard," give way to the most logical plan, but plans and sketches will be replaced by a physical picture which is at once understood by public officials, representatives, and the public at large. Such a model can serve as a valuable educational medium among the people directly concerned with the project, and, in place of inferences of political pressure or inadequate educational effort, its scientific operation will direct confidence toward the engineers and officials charged with the responsibility of creating the best and most economical St. Lawrence Waterway.

[38] Editorial, *Engineering News-Record*, June 22, 1933, p. 819.

L. F. HARZA,[30] M. AM. SOC. C. E. (by letter).[30a]—The author has done a great service to the taxpayers and power users of both Canada and the United States by his analysis of the proposed official plans for developing water power along the International Section of the St. Lawrence River. Since he has had no previous connection with any of the St. Lawrence River investigations, his analysis is unbiased by preconceived ideas. On the other hand, the analysis may have fallen into error if the unit prices or other basic assumptions of Plan *C* are comparable with Plans *A* and *B*, a question regarding which a difference of opinion exists.

The excess cost and its equivalent, of the two-project plan over either of the single-project plans, is so great, even after allowing for possible error, that exceedingly momentous reasons of absolute vital concern to the success of the project would need to be advanced to justify the extra cost and equivalent capitalized excess operating expense.

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[30] Cons. Engr.; Pres. Harza Eng. Co., Chicago, Ill.

[30a] Received by the Secretary October 16, 1933; these paragraphs should replace the first two paragraphs of Mr. Harza's discussion on this subject in November, 1933, *Proceedings*, p. 1500.

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DISCUSSIONS

ON THE BEHAVIOR OF SIPHONS

Discussion

BY IVAN M. NELIDOV, ASSOC. M. AM. SOC. C. E.

IVAN M. NELIDOV,[5] ASSOC. M. AM. SOC. C. E. (by letter)[5a].—The outstanding feature of this paper is the demonstration of the fact that maximum vacuum occurs under certain conditions, not at the throat, as has been always assumed, but somewhat below the throat. The vacuum is also accompanied by a certain degree of contraction of the cross-section area. If the amount of contraction and the position of the contracted section could be expressed through sufficiently accurate coefficients, the conditions of flow and the efficiency of the siphon could be determined more accurately.

Before proceeding with the discussion of this paper the following constructive rather than critical comment is offered concerning the nomenclature, in order to simplify the terminology. "Summit" as introduced by the author has usually been termed "throat," and since, in most instances, it is a contracted section, it appears that the latter is more truly expressive. In accordance with the definitions of the Special Committee on Irrigation Hydraulics,[6] instead of "invert," it appears that the term, "crest," is more suitable. This is true especially because, during part of the operating time, the siphon functions as an overflow spillway. "Operating head" appears to be a more general definition than "siphon head," independent of the conditions of submergence at the outlet. The symbol, m, for atmospheric head is used by the Special Committee to denote mass. The writer proposes H_a for atmospheric head and H_o for operating head. The notations in this discussion are those of the main paper.

Referring to the theoretical analysis it can be mentioned that the theory of a siphon flowing full can be found in many books on hydraulics or irrigation.[7] In order to account for the flow through a siphon the influence of the

NOTE.—The paper by J. C. Stevens, M. Am. Soc. C. E., was published in August, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

[5a] Received by the Secretary, October 28, 1933.

[6] *Proceedings*, Am. Soc. C. E., May, 1932, p. 729.

[7] "Irrigation Practice and Engineering," by B. A. Etcheverry, M. Am. Soc. C. E., Vol. III, pp. 171–172, 1916.

centrifugal force and the contraction of cross-sections must be introduced. In theoretical investigations equations of uniform flow are used, in which all losses are taken proportional to the second power of velocity. Consequently, the coefficients expressing these losses should take account of the following factors: (1) Loss of head due to turbulent flow, loss of head due to partial filling, etc.; (2) local losses caused by curves, projections, entrance, outlet, etc.; and (3) uneven distribution of velocities across the section.

When a conduit begins to act as a siphon its efficiency as such may be reduced by partial filling and eddying. The necessity of designing it so that the barrel will be completely filled is apparent, and this condition will require smooth curves properly designed. On the other hand, a sharp curve is necessary, at least at the summit, in order to hasten and insure priming. The design requirements for a siphon may be enumerated as follows: (*a*) Priming time of long or short duration; (*b*) high or low priming head; (*c*) high or low rate of rise of forebay; (*d*) high or low coefficient of discharge; (*e*) breaking time of long or short duration; (*f*) protection against débris and ice; and (*g*) importance of eliminating vibrations and vortices. Combinations of these requirements may produce various types of siphons, best adapted to the conditions of the project.

Systematized knowledge necessary for satisfying these conditions is very meager. It is only known generally that: (*a*) Fast priming is produced by a perfect water-seal at the outlet, by sharp outlet curves, by a high rate of rise of the forebay, by a small height of the throat, and by small operating head; (*b*) low priming head is produced by good entrance conditions, by a water-seal that is not extremely high, by small outlet radius, and by small submergence of the discharge lip; and, (*c*) high efficiency is produced by smooth curves, Venturi-shaped siphon, and certain height of water-seal in respect to the operating head.

Most of the existing siphons represent complex hydraulic structures. They have bell-shaped entrances of irregular form, widely varying cross-sections, projections, positive and negative inclinations of the outlet leg, various shapes of outlet, etc. Experiments on hand do not supply complete information for the designer. He is forced to use the same old "Weisbach formula" for losses in curves and to guess other numerical factors. Available data are obtained usually on small scale structures, in which the influence of the force of gravity could not properly manifest itself. There is now an urgent need for quantitative data which could be used in design with a fair degree of certainty. The experiments should supply data not only for siphons running full, but also with various quantities of air carried by flow.[8]

Under present conditions the analytical study of the performance of a siphon is applicable only to the evaluation of discharge. An analytical interpretation of factors other than discharge, contributing to the action of a siphon, such a priming head, priming time, etc., is impossible, because of the many variables involved, or due to the influence of the unstable conditions

[8] "Experimentation upon Model Siphons Sucking Air," Thesis by Messrs. William Saylor and David W. Anderson, Univ. of California, Berkeley, Calif.

of flow. Empirical formulas will probably approach the solution in the form of exponential functions.

The definition of efficiency of a siphon by the author, as expressed in Equation (13), agrees with that of Professor J. N. LeConte.[9] As the author correctly states engineers are interested in discharge efficiency. In the writer's opinion, it will be as follows: For $H > h_m$ + losses:

$$Q_o = C_o \sqrt{2\,g\,H} \text{ and } e = \frac{C_o \sqrt{2\,g\,m\,a_o}}{\sqrt{2\,g\,m\,a_s}} = C_o \frac{a_o}{a_s} = C_s.$$

[9] "Hydraulics," by J. N. LeConte, p. 48, 1926.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

STABILITY OF STRAIGHT CONCRETE GRAVITY DAMS

Discussion

BY MESSRS. A. A. EREMIN, AND CALVIN V. DAVIS

A. A. EREMIN,[14] ASSOC. M. AM. SOC. C. E. (by letter).[14a]—In this paper the author has developed an interesting criterion of stability of the straight gravity dam with consideration of shear resistance. Mr. Henny has pointed out correctly that shear strength in masonry is difficult to determine. The angle of the plane of failure in compression test specimens varies with the physical qualities of the material and the method of transmitting compression forces. Professor A. Morley[15] has shown the failures in rock specimens subjected to direct compression in which the planes of failure are almost in the same direction as that of the compression forces, indicating, therefore, that the specimen has failed from tensile stresses. The compression forces were applied to the specimen through thin steel plates. Similarly, tensile failure occurs in compression tests if steel plates are covered with paraffin.[16] It is almost impossible to obtain a uniform distribution of compressive stresses on the plates and the initial failure usually occurs from tensile stresses.

In Coulomb's shear equation cited by Mr. Henny the elastic properties of material are not considered. Therefore, it may be applied for the determination of shear resistance in brittle materials only. Professor O. Mohr has shown that shear strength determines the elastic properties of material and also that a plane of the maximum shear stresses is equi-distant from the maximum and minimum normal stresses. In practice, a distribution of the maximum shear stresses has been proved by Lüder's lines. At present, valuable facts concerning stress distribution are being revealed by photo-elastic analysis based on the distribution of maximum shear stresses in the elastic materials.

NOTE.—The paper by D. C. Henny, M. Am. Soc. C. E., was published in September, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1933, by H. de B. Parsons, M. Am. Soc. C. E.

[14] Associate Bridge Designing Engr., State Highway Comm., Sacramento, Calif.

[14a] Received by the Secretary October 23, 1933.

[15] "Strength of Materials," by A. Morley, 1917, p. 524.

[16] "Strength of Materials," by Prof. S. P. Timoshenko, Russian Edition, p. 28.

Measurements have indicated the pronounced deformations in straight gravity dams.[17] It remains to be determined whether these deformations are due to slippage or elastic deformations in concrete and foundation material. With sufficient experimental data the elastic method will yield the most economic and safest design. However, the middle third theory, the sliding factor theory, and Coulomb's internal friction theory are convenient guides to the formation of judgment as to the stability of structures.

CALVIN V. DAVIS,[18] M. AM. SOC. C. E. (by letter)[18a].—It is gratifying to note that Mr. Henny has shown the fallacy of considering the sliding factor as a measure of the safety of a gravity dam against down-stream movement. His substitution of a factor of safety against shearing failure for the sliding factor is entirely logical and is in keeping with modern design methods.

The author concluded from certain cylinder tests conducted by the U. S. Bureau of Reclamation "that there is a persistent increase of shearing strength with increasing load." This conclusion, which agrees substantially with Coulomb's theory, is of great importance in the design of both gravity and buttress dams, but applies with particular force to the latter.

These tests and the laws derived therefrom show that in the design of buttress dams the factor of safety against shearing failure would be increased if the second principal stresses have substantial values in compression throughout the buttresses. This is particularly true near the up-stream buttress faces where the critical stresses most frequently occur. This principle has been ignored all too frequently in the design of buttress dams.

The importance of the intensity and distribution of shearing stresses in a buttress dam was also recognized by Dr. A. Stucky,[19] a Swiss engineer, who made a careful study of the causes of failure of the Gleno Dam in Italy. Dr. Stucky concluded that the shearing failure of the buttresses of this multiple-arch dam was a contributing cause of this disaster. While there was considerable controversy over Dr. Stucky's conclusions, the principles of design introduced in his report are of importance and are intimately related to those developed by Mr. Henny.

An important relation exists between shearing stress intensity and distribution and the structural safety of dams. In using modern designing methods in the proportioning of buttress dams the writer has experienced no difficulties in obtaining a substantially uniform distribution of the first principal stresses, the vertical normal stresses, and the shearing stresses on any horizontal plane. Such stress distribution gives both maximum economy and maximum safety. If a buttress dam is proportioned for uniform shearing stresses on any horizontal plane, Equation (2) is correct for a value of S equal to the actual shearing strength of the foundation material. No reduction factor such as those which Mr. Henny has included in his equations will be necessary to take care of the high localization of shearing stresses such as occur at the down-stream edge of a gravity dam.

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[17] *Civil Engineering,* August, 1932, p. 489.

[18] Chf. Designer, Ambursen Dam Co., Inc., New York, N. Y.

[18a] Received by the Secretary November 24, 1933.

[19] *Engineering News-Record,* March 20, 1924, p. 486.

The trend toward the use of inclined joints in both gravity and buttress dams makes consideration of the shearing stresses all the more important. If such joints are used the factor of safety against shearing failure of the down-stream column formed by the joint should be computed independently of the remainder of the structure. In the straight gravity type of dam the highest shearing stress intensity would occur in this column, and its factor of safety would be much less than that of the dam as a whole.

In regard to sliding factors, the writer's experience has been that if dams of both the gravity and buttress type are conservatively designed in respect to the intensity and distribution of the principal and shearing stresses, the sliding factor will have a value that is well within accepted limits. For example, in a recently completed design, for a buttress dam, 400 ft high, the sliding factors decreased from a value of 0.75 at a height of 200 ft to 0.64 at a height of 400 ft. The maximum first principal stress in this design was maintained at a uniform value of 450 lb per sq in. throughout the entire structure, and the second principal stress had substantial values in compression at the critical points. The writer has been called upon frequently to analyze designs for dams having sliding factors much higher than those allowed by accepted practice. In all cases, an investigation of these designs revealed that the first principal stresses were far from conservative, the second principal stresses had high values in tension, and the shearing stresses were excessive.

The author is correct in stating that drains in a gravity dam can not be depended upon to eliminate uplift. In view of the great base length required to provide safety against both uplift and shear for high dams, as demonstrated by Mr. Henny, the writer believes that economy will require the use of modified types of gravity dams in which uplift is positively eliminated. A number of such types were developed by the late Fred A. Noetzli, M. Am. Soc. C. E.[20] These consisted of gravity dams containing sufficient cellular spaces to provide complete drainage. These cellular spaces are of such size that clogging would be impossible.

[20] *Engineering News-Record* December 4, 1930, p. 884.

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DISCUSSIONS

ESTIMATING THE ECONOMIC VALUE OF PROPOSED HIGHWAY EXPENDITURES

Discussion

BY MESSRS. W. S. DOWNS, AND GEORGE E. MARTIN.

W. S. DOWNS,[11] M. AM. SOC. C. E. (by letter)[11a].—The economic value of proposed highway expenditures merits more consideration than the subject has generally received during the hasty construction of road systems in the United States. Indications are that future highway improvements will receive more careful analyses to determine the economic justification of such expenditures, in order that the tax dollar, which has become more difficult to produce, will accomplish the greatest possible benefit. As a result of his careful research and studies in highway economics, Dean Agg has repeatedly pointed the way to sound principles that are an aid to the highway engineer in solving such problems.

His method of determining the annual cost of highways is generally accepted, although there is some confusion concerning the propriety of including an annual interest charge on the public investment when such funds are derived from taxes. Assuming that the motoring public has paid a substantial part of the capital cost of the highways, or that the capital investment comes from general funds raised by taxation, to which the motor-vehicle owner contributes, the question is asked: "Should the motor-vehicle owner be charged interest on funds which he himself contributes?" Dean Agg takes the position, correctly, that proper account of interest must be introduced if strictly correct conclusions are to be drawn His explanation in this respect is satisfying.

Care must be taken in evaluating Equation (3), not to place too great values on S, the salvage value, and n, the number of years the improvement will serve economically. Experience has demonstrated that the economic life expectancy of highway improvements may be easily over-estimated; not

NOTE.—The paper by Thomas R. Agg, M. Am. Soc., C. E. was presented at the meeting of the Highway Division, Atlantic City, N. J., October 10, 1932, and published in September, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1933, by Messrs. W. W. Crosby, Roger L. Morrison, and Samuel B. Folk.

[11] Cons. Engr., Morgantown, W. Va.

[11a] Received by the Secretary November 13, 1933.

so much on account of the enduring qualities of the structures as to the changing needs of traffic. Many improvements have become obsolete by reason of necessary changes in location or design due to the increased speed or in the weight and size of the vehicles.

Likewise, the forecast of traffic should be made conservatively. Annual traffic surveys, with comparatively few exceptions, show a pronounced flattening of the curve of increase within recent years. Many important highways show considerable decreases in the use of the roads. That such trend is not due wholly to a temporary decrease in automobile use is indicated by the fact that it was noticeable in many sections as early as 1928 and 1929, during very prosperous years. It is probably the natural sequence of the extensive increase in the mileage of improved highways, which permits the traffic to distribute itself more widely over parallel routes. The approach to a condition of congestion on any highway will cause the traffic to seek other routes quite as much as a poorly paved surface, bad alignment, or greater distance. The liquidity with which traffic seeks the route of least resistance to its convenient flow makes uncertain the forecast that it will continue to use the highway under consideration when, and if, other competing roads are improved.

Since so many factors enter into the cost of vehicle-operation it would appear impossible to derive a formula that will serve for all highway improvements. Dean Agg more or less confines his discussion to such factors as may have a direct bearing upon vehicle-operation costs, as they relate to various kinds of road surfaces. For this purpose he uses the road classifications, "high type, intermediate type, and low type," to express comparatively the physical characteristics of the surface. It is unfortunate that a more definite classification cannot be suggested that would better define the surface properties that make for economy of vehicle-operation. The all-important element in this respect would seem to be a smooth, firm surface that will lower the tractive resistance. Perhaps there are others; but roughometer tests will indicate that the surface of the so-called intermediate and low-type highways may often be as smooth as those that are generally considered to belong to the high type.

There are other elements of highway improvement, however, quite as important in determining vehicle operation costs as the type of surface. Alignment, as well as grade, requires consideration. Poor alignment not only creates accident hazards, but it reduces the speed of vehicles. The grade, likewise, not only affects gasoline consumption and such other factors of cost related to the mechanical operation of vehicles, but it also reduces the speed of traffic. This suggests a careful investigation of the time element as it may influence the time value of the driver and the occupants of vehicles and as it may be the contributing cause of added mechanical costs due to the less efficient speeds of the faster moving vehicles.

The average present-day passenger car may negotiate a grade of 7 or 8% in direct drive with no appreciable loss of speed, and, consequently, it experiences little loss of efficiency on such grades. The heavy truck, however,

may find it necessary to change to a lower gear ratio on grades of 3 or 4 per cent. In addition to the economic loss suffered by the truck due to the gear change, the slower speed with which it travels imposes on all other traffic on the highway the necessity of either passing such vehicle or reducing the speed to that of the truck. If the alignment is such as to restrict sight distance, or if intervals between vehicles approaching in the opposite direction are not great, the passing of a slow-moving vehicle (which is generally of excessive length), becomes impossible. The loss of car-time, therefore, and the necessity for operating at inefficient speeds are ineconomies imposed upon all traffic.

A remedy for this condition may lie in the construction of four-lane highways which will permit the segregation of the slow-moving vehicles to the outer lanes, thus reserving the inner lanes to the faster moving cars. This, however, entails highway costs that can only be justified economically by a great volume of traffic, such as does not, at present, exist on many highways.

In most studies involving the economics of highway expenditures, it is necessary to consider the motor vehicles as divided into two classes: First, that of the private passenger car, and vehicles of like weight and size which possess common characteristics (this class is sometimes referred to as the "basic type"); and, second, that class composed of the so-called commercial vehicles which vary widely as to size, weight, and engine performance.

In his studies, Dean Agg considers the automobile (or basic type) to include all commercial vehicles with a capacity of 1 ton or less. The commercial class, he considers, is made up of commercial vehicles having capacities of 1 ton or more.

It is difficult to co-ordinate these classes to highways in common use. Highway improvements that are necessary for the efficient operation of one class of vehicles may scarcely benefit the other class; or, the benefits to be derived by one class can be measured only by the removal of restrictions that have been imposed by the presence of the other class on the highways.

It is questionable, therefore, if in determining the economic value of certain highway improvements, it is proper to assume that all classes of vehicles will share alike in the benefits of such improvements either on the basis of the vehicle-mile or the ton-mile. Moreover, the wide differences in the requirements of the two classes of vehicles and the variation in the character of the traffic in different localities precludes such studies being made on the presumption of an average vehicle.

This suggests the desirability of research and study in the economy of providing distinct and separate highways for the two major classes of vehicles, especially where the volume of commercial traffic would appear sufficient to justify such steps.

The basic automobile traffic requires easy curvature, but grade reduction is a secondary consideration. The strength of the pavement will be in proportion to the unit wheel load, and its width (if not reduced from present standards), will offer more convenience and ease of travel. The commercial

traffic requires less refinement in the highway alignment, but demands easy grades. The strength of the pavement, likewise, must be in proportion to the heavier wheel load.

A final thought in connection with this subject relates to the financing of the proposed highway improvements. It is not enough to show that the required expenditures are economically justified in that the savings in vehicle operation costs will exceed the cost of the improvements. Further consideration is required as to whether or not such savings can be capitalized to finance the improvements. Notwithstanding the economic value of highways, expressed as savings in vehicle operation costs, it remains a fact that motor vehicle imposts, thus far, fall far short of paying all the costs of the highways. Highway funds have been (and still are, in part), derived from other sources, such as special assessments and property taxes. The assessment of part of the cost to properties may generally be justified for city streets and roads of little general use, on the theory that special benefits accrue to the property owners; but if the improvements are made solely for the benefit of the highway users in the sense that a low-type surface is replaced with a higher type, or that grade reductions or alignment changes are made in order to reduce vehicle operation costs further, then it is extremely difficult to show that additional benefits will accrue to property owners. When property is taxed for purposes from which it derives no benefit, the value of such property depreciates. This practice is detrimental to the entire tax system, since it subsidizes one class at the expense of another.

Hence, as a final test of the economic value of such improvements to the State, assurance is needed that sufficient additional revenues will be derived from the owners of vehicles, who profit from the improvements, to compensate the State for such cost.

George E. Martin,[12] Assoc. M. Am. Soc. C. E. (by letter)[12a].—The method of obtaining the comparative cost of highway transportation per vehicle-mile on a specified highway, as developed by Dean Agg in this paper, is probably correct, but the use of the old cost data (see Table 1) originally published in 1928,[3] may lead to serious errors when applied to present-day road surfaces.

The relative cost of operating an imaginery "average" automobile on various classes of road is shown in Table 1. The highway engineer who attempts to use this table will substitute for the terms, "high type," or "intermediate type," or "low type," the particular road he has in mind. The first and last terms are fairly clear, but what is to be included in the "intermediate type"? Dean Agg does not define this term in the paper, although elsewhere[3] he has defined roads as follows: (1) High types (pavements of all kinds in average good condition); (2) intermediate types (gravel, macadam, and bituminous-treated surfaces); and (3) low types (natural soil roads and light gravel or sand-clay surfaces).

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[12] Cons. Engr., The Barrett Co., New York, N. Y.

[12a] Received by the Secretary November 14, 1933.

[3] "Operating Cost Statistics of Automobiles and Trucks," by T. R. Agg and H. S. Carter, Members, Am. Soc. C. E., *Bulletin 91*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, July 25, 1928.

On the basis of these definitions, engineers have often assumed that the cost given under "intermediate types" (Table 1) would apply to all roads not specifically included in the other two classes. This assumption is not correct. As a matter of fact, these values apply to gravel roads and not to bituminous surfaces or to the mixed-in-place types of road surface. Many of the data in Table 1 were collected in Iowa and practically all the roads in that State, except city pavements, prior to 1928 were earth, gravel, or concrete. Nearly all the test runs were made on these three types, and the writer has been able to find no published data prior to 1928 on bituminous surface treatment, and only a few on bituminous macadam.

The operating costs shown to vary between types are gasoline, maintenance, depreciation, and tires and tubes. Dean Agg has made the statement[13] that maintenance cost is assumed to vary with fuel consumption and one-half the depreciation is also assumed to vary with fuel consumption, the remainder being due to obsolescence. Assuming that the cost of fuel is 1.00

TABLE 2.—DATA PERTAINING TO OPERATING COST

Item No.	Type of pavement	Cost of operating automobiles at from 25 to 35 miles per hour, in cents	Relative fuel consumption, cars with pneumatic tires, operating at 35 miles per hour	Gasoline Consumption: In miles per gallon	Gasoline Consumption: Comparative value	Tire Wear, in Pounds per 1 000 Miles: McNown	Tire Wear, in Pounds per 1 000 Miles: Dana, 1924–1925	Tire Wear, in Pounds per 1 000 Miles: Dana, 1926
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Concrete	10.00	1.12	14.2	1.00	1.43	0.093	0.097
2	Gravel	11.80	1.36	13.1	1.08	0.78	......	......
3	Bituminous-treated gravel	.....	......	13.9	1.02	......	......	......
4	Oil-treated crushed rock	.....	......	......	......	......	......	0.027
5	Crushed basalt macadam	.....	......	......	......	......	0.476	......
6	Crushed rock macadam	.....	......	......	......	......	......	0.372

for the high-type pavements, Dean Agg gives the comparative fuel consumption for the intermediate type and the low type as 1.20 and 1.47 units, respectively. These are the constants he has used in preparing Table 1 of the paper.[14] Relative fuel consumption, therefore, is the basis for these varying costs, and to check the supporting data, it is necessary to refer to a previous paper[15] by Dean Agg and Harold S. Carter, M. Am. Soc. C. E., in which the relative (yearly average) costs of vehicle operation on roadway surfaces of average Portland cement concrete and ordinary gravel are given (see Table 2, Column (3)). These costs check exactly with the relative cost factor, 1.18, for the intermediate types of roadways in Table 1 of the paper. Dean Agg and Professor Carter[16] also present a tabulation that gives the relative fuel

[13] *Bulletin 91*, Eng. Experiment Station, Iowa, State Coll., Ames, Iowa, July 25, 1928, p. 17.

[14] *Loc. cit.*, Table 5.

[15] "Highway Transportation Costs," by T. R. Agg and H. S. Carter, *Bulletin 69*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, July 9, 1924, p. 20.

[16] *Loc. cit.*, p. 19.

consumption on road surfaces of good Portland cement concrete and gravel, the latter approximated from the Iowa yearly average (see Table 2, Column (4)). This produces comparative values for concrete and gravel roads of 1.00 and 1.22, respectively, which check quite closely with the high-type and intermediate-type values cited previously herein.

Table 1 was evidently correct for gravel in 1924 when the data were collected, but a quite different ratio between concrete and bituminous-treated gravel can be deduced from the results reported by R. G. Paustian, Jun. Am. Soc. C. E., in 1932.[17] The Highway Research Board has published curves[18] which show the effect of road surface on gasoline consumption. Interpolating from these curves for a speed of 35 miles per hr, the results in Table 2, Columns (5) and (6), are obtained for good dry concrete, dry gravel, and bituminous-treated gravel, surfaces.

This would indicate that the values in Table 1 based on a constant of 1.20 between the high type and the intermediate type of surface are certainly not correct for a comparison between concrete and bituminous surface treatments.

The other variable cost is that of tire wear which, according to Dean Agg,[15] is assumed as 1.00 for high-type roads, 2.12 for intermediate types, and 2.90 for low types. Data on this subject have been published by William C. McNown, Assoc. M. Am. Soc. C. E.[19] (see Table 2, Column (7)). Professor H. J. Dana, of the State College of Washington, reports[20] the average results listed in Table 2, Column (8). These averages are for tests run in 1924, 1925, and 1926. One test on oil-treated macadam was run in 1926 and is reported[21] as shown in Table 2, Column (9).

Nothing in Table 2 would indicate a differential in tire wear between concrete and bituminous surface treatments of 2.12. The writer believes that available data are insufficient to assume that the values in Table 1 for "intermediate types" can properly be applied to road surfaces which the average highway engineer considers intermediate types, such as bituminous surface-treated gravel and mixed-in-place surfaces. The values in Table 1 are not averages for the entire range of intermediate types, but apply only to plain gravel. Additional data will be necessary before true values for these intermediate types can be obtained, and while these data are being collected, it is manifestly unfair to apply the values of Table 1 to intermediate types other than plain gravel.

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[17] *Proceedings*, Highway Research Board, December 1–2, 1932.

[18] *Proceedings*, Twelfth Annual Meeting, Highway Research Board, p. 87.

[19] *Proceedings*, Sixth Annual Meeting, Highway Research Board, 1926, Table 1, p. 65.

[20] *Engineering Bulletin No. 18*, State Coll. of Washington.

[21] *Loc. cit.*, p. 14.

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DISCUSSIONS

THE SURVEYOR AND HIS LEGAL EQUIPMENT

Discussion

BY CHESTER MUELLER, ASSOC. M. AM. SOC. C. E.

CHESTER MUELLER,[38] ASSOC. M. AM. SOC. C. E. (by letter)[38a].—A seldom mentioned but vitally important phase of surveying is described in this paper, namely, its legal aspects. In a limited space Professor Holt has struck to the heart of each of the major legal problems of a surveyor with clear, concise explanation that permits of no criticism and leaves nothing to be added except some general comments.

That more time should be spent, in engineering courses, on those fundamentals of law that are closely interwoven with the technical professions is conceded by most engineers. The writer has found that a knowledge of law in engineering work is a valuable asset in many respects.

Professor Holt lays stress on the judgment that a surveyor should exercise in order to do his client (and adjoining land owners) justice. To any one who has served his apprenticeship under a surveyor of the "old school," the meaning is plain. Frequently, a surveyor capable of running lines with precision fails to delineate boundaries correctly, due to such lack of judgment. When such a man has seen a more experienced surveyor define those same boundaries he feels a strange sense of uncertainty in boundary work because so much depends on elements other than a correctly graduated tape and accurately adjusted transit. Surveyors engaged in re-running old lines do well to proceed with caution before insisting to their clients that their work and not the preceding work should be given credence.

When physical signs on the ground run counter to even the most precise survey it ill behooves a surveyor to dismiss them from consideration without first subjecting them to a careful analysis. Litigation and disputes between former friendly neighbors is often due, as Professor Holt observes, to the inexperience of a surveyor. When differences appear between adjoining surveys, that cannot be dissolved by the surveyor last on the ground, it is the

NOTE.—The paper by A. H. Holt, M. Am. Soc. C. E., was published in September, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1933, by Messrs. Ray H. Skelton, William Bowie, R. Robinson Rowe, and Walter H. Dunlap.

[38] Prin. Asst. Engr., Dept. of Public Affairs, and Attorney-at-Law, Newark, N. J.

[38a] Received by the Secretary November 8, 1933.

general practice for him to consult his predecessors and by each surveyor contributing his knowledge of the facts in the case a resulting line is determined that renders justice to each of the abutting land owners. Perhaps, in a measure, the success of such informal arbitration in New Jersey accounts for the relatively few reported boundary cases in recent years. This is all the more unusual because as one of the thirteen original States, New Jersey's land grants extend back to 1664 and were never subject to a systematic geodetic control. In the many years since those early days more refined methods of surveying were naturally grafted on to many of the old lines established by descriptions drawn in Colonial days, with resulting complications.

Two cases, one as recent as 1933, hearken back to pre-revolutionary days and furnish typical examples of how the new lines are interpreted in view of the old. The case of Warner *et al v.* Smith *et al* (164, Atl. 282, 112 N. J. E., 152) required a decision based on a grant made on February 17, 1742, by the Proprietors of West Jersey which covered three islands and which described them by name and acreage only, and on a grant made by the same Proprietors in 1776 of an island described by metes and bounds. The established rule of law, which holds metes and bounds to be controlling over quantity, was followed in this case, but the application of that rule appeared somewhat difficult due to the fact that to-day only three islands remain, whereas the two grants combined called for four. Furthermore, one of the three islands first granted was described as Pork Island and called for 17 acres, whereas the second grant describing Beach Island by metes and bounds and calling for 244 acres, was claimed to apply to the island which to-day is called "Pork Island."

The claimants under the 1742 grant were successful in showing that the acreage reported under that grant was unreliable, but were unable, however, to demonstrate that an island granted as 17 acres in 1742, was the same one granted as 244 in 1776 (accurately surveyed as 312 in 1841); nor were they able to disprove the defendants' evidence that Beach Island first assumed the name of "Pork Island" by popular usage many years after 1776. The case is principally of historical interest to surveyors, but also shows how the fundamental boundary issues may be obscured as a case develops.

The other case decided in 1889 was that of Scott *v.* Yard (18 Atl. 359, 46 N. J. E., 79) and brings out an interesting situation in which a claimant attempted to "take up," legally, what he insisted to be unappropriated land. Scott, the plaintiff, acquired on August 26, 1880, three lots of land fronting on Ocean Avenue, in Spring Lake, N. J., each described as being 50 ft in frontage by 150 ft in depth. His land lay on the west side of Ocean Avenue, the Atlantic Ocean bordering on the east side of the Avenue. Running back Scott's title, two tracts are involved. The west part of his lots were in the Forman tract which was surveyed and allotted to Forman in July, 1746, and the east portion of his lots were in the Brinley tract which was surveyed and allotted on October 1, 1860. The defendant claimed title to a strip crossing Scott's lots and lying between the two tracts mentioned, by reason of an

including survey (approved by the Surveyor-General on October 19, 1800) the nature of which was to embrace land previously surveyed and appropriated by other persons and to "pick up," in the language of the Court, any land unappropriated.

The Court held that the Brinley survey of 1860 contemplated including all the land between the ocean and the east line of the Forman survey, and only by mistake would a surveyor fail to include all the land possible for his client. In this connection, it ruled that surveyors of the past did that which an ordinarily skilful, proper discharge of their duty to their principals required them to do.

Interestingly enough, the Court also brought out the fact that the rules of the Board of Proprietors prohibited its surveyors from fixing locations and lengths of lines by monuments and directed them not to mention monuments in the description of lands they surveyed, except at the beginning point.

An examination of the reported cases which discuss the surveyor as a witness, and what may be evidential in determining land boundaries, will throw considerable light on the opinion in which the Courts hold the functions of a surveyor. The only boundary case reported in New Jersey in 1932[89] contributes something in this respect. The case came to the Supreme Court on the defendant's argument that the verdict rendered in the Circuit Court was against the weight of evidence and that, therefore, a new trial should be granted. In its opinion the Supreme Court stated that "it suffices to say that the testimony of plaintiff's surveyor, a man of many years experience, was subject to credence by the jury and was not overcome by any evidence of a convincing character offered by defendants," and dismissed the defendant's rule.

While informal arbitration and existing statutes are deemed of value, the surveyors of New Jersey have recently been urging some more suitable means, fully countenanced by the law, to adjust boundary disputes and irregularities and to prevent their occurrence. The Legislative Committee of the New Jersey Association of Professional Engineers and Land Surveyors has been giving considerable study to the problem of clarifying and strengthening the law in this respect. Preparation of descriptions by competent surveyors following uniform regulations, filing and recording of survey plans, and properly fixing property boundaries by monuments, are typical of the subjects embraced in the study.

Outstanding in its treatment of land boundary matters is the State of Massachusetts which has a Land Court, that among other duties has exclusive original jurisdiction of petitions to determine the boundaries of lands. The act (Chapter 185 of the General Laws of Massachusetts) which established the Court also provides for the registration of the title to land.[90] The act creating the Court, and the Court's published instructions, furnish excellent material for a basis upon which to redraft legislation in other States.

Searching for the "footsteps of the surveyor," of which Professor Holt writes, is essential to the proper exercise of surveying judgment. When

[89] Girard *v.* Stager, 156 Atl. 14, 9 N. J. Misc., 871.
[90] *Civil Engineering,* August, 1932, p. 506.

working in the field with his father, the late Carl Mueller, who spent a lifetime in the profession, it was a constant source of revelation to the writer how unerringly he sought out and found such "evidence," with a method appropriate to the locality. The result of discriminating evaluation of such evidence was that frequently in driving a corner stake it would hit on a buried remainder of its early predecessor. As aptly expressed by Professor Holt, the cardinal principle is not to correct an established boundary line, but to reproduce it on the ground, and perhaps no better method can be followed than to attempt to retrace, both mentally and physically, the first surveyor's footsteps.

Admitting that surveyors should be familiar with the legal principles encountered in their daily work, it is not amiss to point out that many, if not most, lawyers are totally uninformed as to the nature of the work of a surveyor. They, too, believe that a surveyor following a description (written loosely, or otherwise, by a layman or by a lawyer, more often than a surveyor) should establish the boundaries without effort and with certainty. They do not, as a rule, know what temperature changes do to tapes, why compass bearings vary, and how the many other factors may introduce differences in the surveys of the same boundaries at different times. Above all, they do not realize that the value of land a century or more ago did not warrant the accuracy called for to-day and, therefore, they do not interpret the results accordingly. If surveyors in those cases of necessary contact with lawyers would always keep in mind that they must first "educate" the lawyer in fundamentals before they can hope to have him understand the specific case at hand, they would more readily establish mutual respect and understanding.

As an illustration of the common belief in "measure" as absolute, there is the interesting argument advanced by counsel in a case[91] decided in 1881, a time when most principles of law concerning boundaries in the United States were already well established. The case involved the survey of a boundary line that differed from another survey by 0.29 in. for each 50 ft of length, attributed to the difference in length of the respective 50-ft chains used. An iron monument was found at the disputed corner, but nevertheless the attorney for the defendant argued that since the United States had fixed a standard of length by law, that standard and no other controlled the distance measured along the boundary. In answer, the Court held that a distance gave way to a monument, that a standard unit of measure was not relevant to the issue, and that in its opinion no statute could constitutionally cause boundaries to be determined by measure only.

While the principles of law as applied to land boundaries may be clear, it is their application that presents the most difficulty. For example, a lot was sold by description, as beginning at a marble monument set in the side line of a street. Actually, the monument was a few feet removed from the side line of the street. Analyzing the wording of the description to discover the apparent intent, the Court held that the street itself was a monument

[91] Kalbfleisch *v.* Standard Oil Co., 43 N. J. L., 259.

and as a "natural" one took precedence over the "artificial" monument and that, therefore, the beginning point of the course lay in the side line of the street—a decision that really discredits the original survey, but actually carries out the intent of the parties.

Another instance where intent was carefully scrutinized and where a distance gave way to a physical boundary arose in a case[92] decided in 1880. The deed conveyed a house and the lot on which it stood, among a row of houses, and the description in reference to the depth of the lot read, in part, "thence easterly at right angles to Grove Street 80 feet or a fraction more or less * * *". The grantor only owned 65 ft in depth, a fence standing at this distance back from, and parallel to, Grove Street. The Lower Court gave the plaintiff who was the purchaser, the value of an additional 15 ft, but the Supreme Court resting on the admission of the plaintiff that he had viewed the property prior to purchase and had believed the fence to be on the line, reversed the decision. The perusal of such cases as this one serves to enlighten a surveyor considerably on the reasoning supporting the judicial opinions which have established the legal principles involved.

The beacon for New Jersey jurists, embodying the principles of the law concerning boundaries, is perhaps best expressed in the following opinion of Chief Justice Green in a case[93] decided in 1859:

"The rule is well settled that boundaries may be proved by every kind of evidence that is admissible to establish any other fact. Actual occupation, ancient reputation, the admission of the party in possession against his interest, ancient maps and drafts, marked trees, the line of adjoining surveys, monuments erected at or soon after the date of the grant of adjoining surveys, are all admissible for this purpose, and are constantly resorted to fix the boundaries though it conflicts with the courses and distances called for in the deed. The well settled principle is, that practical location is evidence of a mistake in the description."

Chief Justice Green points out that the practice as well as the reason upon which it rests, is stated admirably by Mr. Justice Washington in Connecticut *v.* Pennsylvania (1 Pet. C. C. 511) as follows:

"No gentleman of the profession who is at all conversant with land trials, can be ignorant that the courses and distances laid down in a survey, especially if it be ancient, are never, in practice, considered conclusive, but that, on the contrary, they are liable to be materially changed by oral proof or other evidence tending to prove that the documentary lines are not those actually run. How often we have known reputed boundaries proved by the testimony of aged witnesses and by such evidence established in opposition to the most precise calls of an ancient patent. Such evidence has been constantly received, and distances have been lengthened or shortened without the slightest regard to the calls of the patent. The reason is obvious. It is not the lines reported, but the lines actually run by the surveyor which vest in the patentee a title to the area included within those lines. The survey returned on the patent is the evidence of the former, natural marks or reputation is in almost all cases the evidence of the latter. The mistakes committed by surveyors and chainbearers, more particularly in an unsettled

[92] Smith *v.* Negbauer, 42 N. J. L., 305.
[93] Opdyke *v.* Stephens, 28 N. J. L., 83.

country and wilderness, have been so common, and are so generally acknowledged as to have given rise to a principle of law as well settled as any which enters into the land titles of this country, which is, that when the mistake is shown by satisfactory proof, courts of law, as well as of equity, have looked beyond the patent to correct it."

Professor Holt's paper serves to point the way to more careful boundary surveys and should encourage those of the profession engaged in such work to a further study of the two reference texts he cites.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

DURATION CURVES

Discussion

BY MESSRS. RICHARD PFAEHLER, AND EDWARD H. SARGENT

RICHARD PFAEHLER,[8] M. AM. SOC. C. E. (by letter)[8a].—The application of the duration curve to hydraulic problems has been clearly presented in this paper. The engineer who is called on to make a report on the feasibility of an hydro-electric, or a water supply, project must devote a considerable part of his allotted time to the collection and consequent analysis of all available stream-flow data with reference to the project; and then he must present his conclusions in such form as will make them easily understood by his client. To this end the graphic method, including hydrographs, storage curves, mass curves, and flow-duration curves, as described by the author, is often used to supplement any analytical data and tabulations.

In 1908 and 1909 the writer made a stream-flow study of the principal streams in North Carolina and South Carolina. At that time (which coincided with the beginning of the intensive development of the cotton-mill industry in the South), it was customary to develop the "run of the river," and the general rule was that a power installation corresponding to an eight months' flow was practical and economical. This, of course, suggested the preparation of flow-duration records with the use of flow-duration curves, which were usually plotted with the abscissa giving the percentage of time that the flow is equal to, or more than, any given rate. Due to the great irregularity in flow of these streams only the daily duration curves were used.

As pointed out by Mr. Foster, it is sometimes necessary to use monthly duration curves, due to the meagerness of the available stream-flow data, or due to the limited time allowed for the study. In many reports reviewed by the writer the monthly curves were used as a basis in arriving at the estimated power output, and in checking the figures furnished in such a report by the use of the daily stream-flow records, the results thus obtained sometimes differed as much as 35% to the disadvantage of the project. This last-named

NOTE.—The paper by H. Alden Foster, M. Am. Soc. C. E., was published in October, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[8] Design and Cons. Engr., Duke Power Co. and Allied Interests, Charlotte, N. C.

[8a] Received by the Secretary November 6, 1933.

figure refers to rivers of the flashy type. Now, it is often quite true that the hydraulic engineer is not in a position to spend the time required for compiling, tabulating, and plotting the daily values. If this is the case the engineer should be aware of the probable percentage of error introduced by the monthly curves and should make proper allowance in his final estimates and conclusions.

It is aptly stated by the author that the duration curve represents average annual conditions for a number of years or, in other words, that it is a curve for a typical year. In this connection it is suggested that a distinction be made between a "typical flow-duration curve" and a "low-flow duration curve." The latter is meant to cover years of extreme low-water flow only, such as occurred, for instance, in the Southern States in 1904 and 1925. By this means a clear picture is presented of what may be expected in such periods of stress. This applies, of course, to both the natural and the regulated flow curve.

A few words on the use of the duration curve of regulated flow: As it is known, the magnitude of the minimum regulated flow depends on the available useful storage of the reservoir for the project under consideration. There is no doubt that by means of the mass curve, or by some method of tabulation of stream-flow records, the minimum regulated discharge can be determined conveniently for any year or critical season, provided the records of natural flow cover a sufficiently long period. With this information at hand, the engineer can prepare a schedule of regulated flow which, in his opinion, is best suited to the project. The writer is inclined to think that the duration curve of regulated flow will introduce difficulties when it is applied to complicated stream-flow and reservoir studies; but, for the "run-of-river" projects, the use of the duration curve is to be highly recommended.

In conclusion, it is to be hoped that Mr. Foster's able paper on the application of the duration curve, and a knowledge of its limitations, will be of great assistance to the engineer of less experience in this line of endeavor; and that it will enable him to steer the proper course in the solution of hydraulic problems of this nature.

EDWARD H. SARGENT,[9] M. AM. SOC. C. E. (by letter).[9a]—The author is to be commended for explaining in simple terms the preparation and analysis of duration curves.

The writer has written a description elsewhere[10] of the method of preparing duration curves, including about forty curves of the principal streams in New York State. Several of these curves were computed on a daily as well as on a monthly basis similar to that indicated by Mr. Foster.

The duration-area curve described by the author is a most helpful device. A somewhat similar method was used by the writer[11] to obtain a so-called "energy curve" by planimetering the area within the duration curve and below

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[9] Chf. Engr., Hudson River Regulating Dist., Albany, N. Y.

[9a] Received by the Secretary October 20, 1933.

[10] Annual Rept., New York State Conservation Comm., 1919, p. 463 *et seq.*

[11] An illustration of such a curve will be found on Pl. 12, Annual Rept., New York State Water Supply Comm., 1911.

a given flow or capacity line, depending on whether the ordinate was in terms of flow or power.

Mr. Foster's illustration of the application of the duration curve to hydro-electric problems is well done. However, it has been the writer's experience in analyzing the output of hydro-electric plants that the power indicated to be available by duration curves should be discounted considerably unless the output is fed into a large system capable at all times of absorbing the capacity of the plant.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from December 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

*** Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.**

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ALBERT, JAMES GEORGE, New York City. (Age 24.) Draftsman, Aerial Photo Compilation Unit, U. S. Coast & Geodetic Survey. Refers to E. F. Berry, L. Mitchell, S. D. Sarason.

BAIHLY, WILLIAM EDINGTON, Tacoma, Wash. (Age 28.) Refers to I. L. Collier, C. W. Harris, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

BAYOT, JEAN MARIE, Caracas, Venezuela. (Age 32.) Gold prospecting. Refers to E. Aguerrevere, P. L. Hornby, E. G. Joubert, W. S. Merrill, E. Ortega-Rosado.

BEEBE, HAROLD DeFOREEST, North Plainfield, N. J. (Age 24.) Refers to C. T. Bishop, C. S. Farnham, P. G. Laurson, R. H. Suttie, J. C. Tracy.

BIEMANN, BERNHARD FREDERICK, Richmond Hill, N. Y. (Age 33.) With Home Title Guaranty Co., Brooklyn, N. Y. Refers to J. J. Durfee, C. S. Gleim, R. W. Greenlaw, W. M. Griffin, M. I. Killmer, H. L. King, J. Mechanic, H. F. Pfau, C. T. Schwarze, F. A. Snyder, J. M. C. van Hulsteyn.

BRINGHURST, JOHN HENRY, Jr., Houston, Tex. (Age 20.) Refers to G. H. Lacy, J. Lansdale, L. B. Ryon, Jr.

BROWN, CHARLES REED, Monroe, La. (Age 30.) Chf. of Party, U. S. Engrs. Refers to O. G. Baxter, F. J. Brown, G. R. Clemens, L. F. Reynolds, K. R. Young.

BURLAND, CARLYLE GRAY, Leominster, Mass. (Age 28.) Chf. of Survey Party, Massachusetts Dept. of Public Works. Refers to R. E. Coomes, A. W. Dean, R. K. Hale, G. E. Harkness, J. A. Johnston, A. E. Kleinert, Jr.

CAMPBELL, RAY ANDERSON, Casper, Wyo. (Age 25.) Rodman, U. S. Bureau of Reclamation. Refers to R. D. Goodrich, E. K. Nelson, H. T. Person.

CAROLLO, JOHN ANDREW, Phoenix, Ariz. (Age 27.) Junior member, Headman, Ferguson, Engrs. Refers to J. A. Fraps, S. A. Greeley, P. Hansen, H. A. Olson, J. H. Rider, W. E. Stanley.

FARNEY, HAROLD SAMUEL, Castorland, N. Y. (Age 23.) Refers to C. M. Spofford, F. C. Wilson.

FULLER, WILLIAM JOHN, Milwaukee, Wis. (Age 51.) Prof. of Civ. Eng., Univ. of Wisconsin. Refers to D. W. Mead, J. P. Schwada, F. E. Turneaure, L. F. Van Hagan, C. S. Whitney.

GROSSMAN, EDWARD, Boston, Mass. (Age 35.) Engr. and Archt. Refers to E. F. Allbright, H. B. Alvord, O. H. Horovitz, J. C. Moses, E. A. Varney, B. White.

HALL, CRISPIN CLEMENT, Buffalo, N. Y. (Age 33.) Jun. Asst. Engr., Dept. of Public Works, Div. of Highways, New York State. Refers to J. J. Phelan, T. M. Ripley, C. M. Spofford, N. H. Sturdy, G. F. Unger.

HEFLIN, CARL WASHINGTON, Stapleton, N. Y. (Age 40.) Field Engr., Constr. Dept., Baltimore & Ohio R. R. Co. Refers to E. J. Dougherty, D. W. Fry, W. W. Gwathmey, Jr., J. W. Jones, H. A. Lane, R. Mather, H. R. Talcott.

HENSHAW, LAMOND FORBES, Portland, Ore. (Age 23.) Jun. Engr., War Dept., Corps of Engrs. Refers to W. B. Bennett, G. H. Canfield, H. G. Gerdes, F. F. Henshaw, A. H. Horton, H. A. Rands, J. C. Stevens.

LEEPER, LAVERNE DAVIDSON, Pasadena, Cal. (Age 23.) Designer with Oliver G. Bowen, Cons. Structural Engr., Los Angeles, Cal. Refers to R. R. Martel, F. Thomas.

MASON, ALVIN HUGHLETT, Harrisonburg, Va. (Age 28.) Computer-Draftsman, U. S. Forest Service, Washington, D. C. Refers to M. J. Bussard, W. R. Glidden, B. P. Harrison, W. E. Rowe, M. S. Wright.

MILES, THOMAS KIRK, Stanford University, Cal. (Age 23.) Refers to H. K. Barrows, J. B. Cox, F. H. Fowler, C. C. Kennedy, L. B. Reynolds.

MORRIS, SETH BRADLEY, Urbana, Ill. (Age 33.) Graduate student, Univ. of Illinois. Refers to C. G. Conley, H. Cross, C. Johnson, W. P. Linton, W. M. Wilson.

O'BRIEN, FRANCIS JOSEPH, Safe Harbor, Pa. (Age 27.) Eng. Inspector, Safe Harbor Water Power Corporation. Refers to C. M. Africa, H. F. Anthony, P. E. Gisiger, P. H. La Rosee, A. A. Meyer, H. E. Whitney.

PAGAN, PEDRO COLON, Villalba, Puerto Rico. (Age 32.) Asst. Engr. with Govt. of Puerto Rico. Refers to E. Baez Rodriguez, M. Font, R. A. Gonzalez, E. S. Jiminez, A. S. Lucchetti-Otero, R. M. Snell, J. Urbino, C. del Valle Zeno.

PROCUNIAR, ROBERT WILLIAM, Dayton, Ohio. (Age 28.) With City of Dayton, Ohio. Refers to G. F. Baker, E. O. Brown, J. F. Hale, M. Hay, W. W. Morehouse.

REES, HAROLD, Cheyenne, Wyo. (Age 26.) Refers to R. D. Goodrich, H. T. Person.

SALISBURY, EUGENE FRANKLIN, Minden, La. (Age 47.) Chf. Engr., Louisiana & Arkansas R. R. Refers to C. N. Kast, C. S. Kirkpatrick, H. D. Knecht, A. A. Miller, R. Owen, R. L. Tatum, W. H. Vance, S. L. Wonson.

SCHOLTZ, WALTER, Los Angeles, Cal. (Age 22.) Asst. Shop Foreman, New England Pacific Screw Metal Products Co. Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas.

SHEARER, HERBERT LEWIS, Lexington, Ky. (Age 42.) With Central Rock Co. Refers to W. J. Carrel, J. F. Grimes, J. W. Guyn, R. B. Hayes, D. V. Terrell, J. S. Watkins.

SUTER, MAX, Urbana, Ill. (Age 44.) Graduate student, Univ. of Illinois. Refers to O. H. Ammann, H. E. Babbitt, J. S. Crandell, J. J. Doland, M. L. Enger, H. F. Ferguson, W. D. Gerber, W. D. Hatfield, W. C. Huntington, G. W. Pickels.

TAYLOR, JAMES ELLIOTT, Seminole, Tex. (Age 28.) Res. Engr., Texas Highway Dept. Refers to O. V. Adams, G. R. Johnston, H. N. Roberts.

TRAVIS, WAYNE IVAN, Boise, Idaho. (Age 25.) Jun. Engr., U. S. Geological Survey. Refers to I. C. Crawford, R. G. Kasel, T. R. Newell, C. G. Paulsen.

VALENSTEIN, SAMUEL, New York City. (Age 42.) Chf. Engr. with M. Shapiro & Son, Engrs. and Contrs. Refers to W. J. Ash, H. G. Balcom, A. J. Bernstein, H. H. Cashdan, N. A. Richards, O. Singstad, D. B. Steinman.

VAN HASSELT JACOB ADRIAAN KAREL, Yellow Springs, Ohio. (Age 51.) Prof. of Civ. Eng., and Acting Head of Dept. of Eng., Antioch Coll. Refers to A. W. Buel, N. C. Grover, S. Johannesson, W. J. Scott, H. Stabler, R. H. Suttie.

VIZCARRONDO, JULIO EUGENIO, Santurce, Puerto Rico. (Age 25.) Contr. Refers to J. Benitez-Gautier, A. S. Lucchetti-Otero, F. Pons, S. Quinones, R. Ramirez, A. S. Romero, E. Totti y Torres, C. del Valle Zeno.

WRIGHT, ROBERT ERNEST, Sierre Madre, Cal. (Age 40.) Engr., Standard Oil Co. of California. Refers to G. Boschke, C. E. Cate, G. R. G. Conway, H. Nunn, E. K. Smoot.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

CRAGGS, CLYDE, Assoc. M., Chicago, Ill. (Elected Jan. 17, 1927.) (Age 36.) Chf. Engr., Worden-Allen Co. and Permanent Constr. Co. Refers to H. H. Brown, G. Bryan, Jr., J. R. Fordyce, E. W. Kelly, C. Older, V. C. Ward, P. A. Zahorik.

HOUGH, FLOYD WOODWORTH, Assoc. M., Banning, Cal. (Elected Junior June 1, 1920; Assoc. M. Oct. 12, 1925.) (Age 35.) Engr., Colorado River Project, Metropolitan Water Dist. of Southern California. Refers to G. E. Baker, E. A. Bayley, J. B. Bond, W. Bowie, R. B. Diemer, C. L. Garner, E. O. Heaton.

KURTZ, ROBERTO, Assoc. M., Canpana, Buenos Aires, Argentine Republic. (Elected Oct. 12, 1925.) (Age 55.) Member of firm, Demartini & Kurtz, Engrs. and Constrs. Refers to R. Dominguez, D. P. Krynine, A. B. Lea, E. Lenhardtson, J. A. Valle.

MEAD, WILLIAM HENRY, Assoc. M., Luling, Tex. (Elected Oct. 9, 1917.) (Age 46.) Chf. Engr. and Gen. Supt., Salt Flat Water Co. and Darst Salt Water Co. Refers to H. Culpeper, R. J. Cummins, J. M. Howe, J. C. McVea, E. N. Noyes, F. H. Shaw.

MONTZ, JOHN McGILL, Assoc. M., Columbus, Ohio. (Elected Feb. 25, 1924.) (Age 48.) Associate Prof. Civ. Eng. Dept., Ohio State Univ. Refers to J. B. Jenkins, H. A. Lane, C. T. Morris, J. C. Prior, J. S. Sewell, C. E. Sherman, R. C. Sloane, E. Stimson.

FROM THE GRADE OF JUNIOR

GROOMS, ROY SEWELL, Jun., Decatur, Ala. (Elected Nov. 14, 1927.) (Age 32.) Designer, Alabama State Highway Dept., First Div. Refers to C. A. Baughman, H. D. Burnum, J. A. C. Callan, H. R. Creal, W. H. Fisher, P. D. Gillham, G. H. Sager, Jr., N. Williams.

HENNY, ARNOLD LORENTZ, Jun., Denver, Colo. (Elected Nov. 10, 1930.) (Age 30.) With U. S. Bureau of Reclamation. Refers to I. E. Houk, H. R. McBirney, B. S. Morrow, J. L. Savage, B. W. Steele, J. C. Stevens.

HOOVEL, WILLIAM HENRY, Jun., Waterbury, Conn. (Elected Oct. 10, 1927.) (Age 32.) Asst. Engr., Bureau of Eng. Refers to A. H. Beyer, R. A. Cairns, T. M. Curry, Jr., J. K. Finch, D. G. Follett, W. J. Krefeld.

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LORENZINI, ERNEST MAURICE, Jun., Belmont, Mass. (Elected Oct. 14, 1930.) (Age 28.) Engr. with American Bitumuls Co., San Francisco, Cal. Refers to H. P. Boardman, P. L. Fahrney, C. L. McKesson, J. C. McLeod, G. P. Morrill, R. M. Morton, J. Moskowitz, A. R. Norcross.

MILLER, GEORGE HODGSON, Jun., Boise, Idaho. (Elected Nov. 14, 1927) (Age 31.) Engr-Examiner (under State Engr.), Federal Emergency Administration of Public Works. Refers to A. D. Butler, I. C. Crawford, C. D. Forsbeck, B. J. Garnett, H. M. Hadley, S. B. Nevius, L. E. Rydell, F. J. Sharkey, A. M. Truesdell, M. Yerington.

ORTOLANI, WALTER ALBERT, Jun., Wharton, Tex. (Elected June 7, 1926.) (Age 32.) Res. Engr., Texas State Highway Dept. Refers to S. D. Bacon, W. O. Jones, J. T. L. McNew, J. M. Nagle, J. J. Richey, M. A. Stainer, F. J. Von Zuben.

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VERANTH, JOSEPH, Jun., Ely, Minn. (Elected April 7, 1930.) (Age 32.) City Engr. Refers to E. F. Brownell, O. E. Brownell, G. H. Butler, Sr., R. W. Crum, J. P. Gebhard, L. M. Martin, S. W. Tarr.

WICKHAM, JOSEPH JOHN, Jun., Scranton, Pa. (Elected June 6, 1927.) (Age 32.) Textbook Writer in School of Civ.,

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ALBERT, JAMES GEORGE, New York City. (Age 24.) Draftsman, Aerial Photo Compilation Unit, U. S. Coast & Geodetic Survey. Refers to E. F. Berry, L. Mitchell, S. D. Sarason.

BAIHLY, WILLIAM EDINGTON, Tacoma, Wash. (Age 28.) Refers to I. L. Collier, C. W. Harris, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

BAYOT, JEAN MARIE, Caracas, Venezuela. (Age 32.) Gold prospecting. Refers to E. Aguerrevere, P. L. Hornby, E. G. Joubert, W. S. Merrill, E. Ortega-Rosado.

BEEBE, HAROLD DeFOREEST, North Plainfield, N. J. (Age 24.) Refers to C. T. Bishop, C. S. Farnham, P. G. Laurson, R. H. Suttie, J. C. Tracy.

BIEMANN, BERNHARD FREDERICK, Richmond Hill, N. Y. (Age 33.) With Home Title Guaranty Co., Brooklyn, N. Y. Refers to J. J. Durfee, C. S. Gleim, R. W. Greenlaw, W. M. Griffin, M. I. Killmer, H. L. King, J. Mechanic, H. F. Pfau, C. T. Schwarze, F. A. Snyder, J. M. C. van Hulsteyn.

BRINGHURST, JOHN HENRY, Jr., Houston, Tex. (Age 20.) Refers to G. H. Lacy, J. Lansdale, L. B. Ryon, Jr.

BROWN, CHARLES REED, Monroe, La. (Age 30.) Chf. of Party, U. S. Engrs. Refers to O. G. Baxter, F. J. Brown, G. R. Clemens, L. F. Reynolds, K. R. Young.

BURLAND, CARLYLE GRAY, Leominster, Mass. (Age 28.) Chf. of Survey Party, Massachusetts Dept. of Public Works. Refers to R. E. Coomes, A. W. Dean, R. K. Hale, G. E. Harkness, J. A. Johnston, A. E. Kleinert, Jr.

CAMPBELL, RAY ANDERSON, Casper, Wyo. (Age 25.) Rodman, U. S. Bureau of Reclamation. Refers to R. D. Goodrich, E. K. Nelson, H. T. Person.

CAROLLO, JOHN ANDREW, Phoenix, Ariz. (Age 27.) Junior member, Headman, Ferguson, Engrs. Refers to J. A. Fraps, S. A. Greeley, P. Hansen, H. A. Olson, J. H. Rider, W. E. Stanley.

FARNEY, HAROLD SAMUEL, Castorland, N. Y. (Age 23.) Refers to C. M. Spofford, F. C. Wilson.

FULLER, WILLIAM JOHN, Milwaukee, Wis. (Age 51.) Prof. of Civ. Eng., Univ. of Wisconsin. Refers to D. W. Mead, J. P. Schwada, F. E. Turneaure, L. F. Van Hagan, C. S. Whitney.

GROSSMAN, EDWARD, Boston, Mass. (Age 35.) Engr. and Archt. Refers to E. F. Allbright, H. B. Alvord, O. H. Horovitz, J. C. Moses, E. A. Varney, B. White.

HALL, CRISPIN CLEMENT, Buffalo, N. Y. (Age 33.) Jun. Asst. Engr., Dept. of Public Works, Div. of Highways, New York State. Refers to J. J. Phelan, T. M. Ripley, C. M. Spofford, N. H. Sturdy, G. F. Unger.

HEFLIN, CARL WASHINGTON, Stapleton, N. Y. (Age 40.) Field Engr., Constr. Dept., Baltimore & Ohio R. R. Co. Refers to E. J. Dougherty, D. W. Fry, W. W. Gwathmey, Jr., J. W. Jones, H. A. Lane, R. Mather, H. R. Talcott.

HENSHAW, LAMOND FORBES, Portland, Ore. (Age 23.) Jun. Engr., War Dept., Corps of Engrs. Refers to W. B. Bennett, G. H. Canfield, H. G. Gerdes, F. F. Henshaw, A. H. Horton, H. A. Rands, J. C. Stevens.

LEEPER, LAVERNE DAVIDSON, Pasadena, Cal. (Age 23.) Designer with Oliver G. Bowen, Cons. Structural Engr., Los Angeles, Cal. Refers to R. R. Martel, F. Thomas.

MASON, ALVIN HUGHLETT, Harrisonburg, Va. (Age 28.) Computer-Draftsman, U. S. Forest Service, Washington, D. C. Refers to M. J. Bussard, W. R. Glidden, B. P. Harrison, W. E. Rowe, M. S. Wright.

MILES, THOMAS KIRK, Stanford University, Cal. (Age 23.) Refers to H. K. Barrows, J. B. Cox, F. H. Fowler, C. C. Kennedy, L. B. Reynolds.

MORRIS, SETH BRADLEY, Urbana, Ill. (Age 33.) Graduate student, Univ. of Illinois. Refers to C. G. Conley, H. Cross, C. Johnson, W. P. Linton, W. M. Wilson.

O'BRIEN, FRANCIS JOSEPH, Safe Harbor, Pa. (Age 27.) Eng. Inspector, Safe Harbor Water Power Corporation. Refers to C. M. Africa, H. F. Anthony, P. E. Gisiger, P. H. La Rosee, A. A. Meyer, H. E. Whitney.

PAGAN, PEDRO COLON, Villalba, Puerto Rico. (Age 32.) Asst. Engr. with Govt. of Puerto Rico. Refers to E. Baez Rodriguez, M. Font, R. A. Gonzalez, E. S. Jiminez, A. S. Lucchetti-Otero, R. M. Snell, J. Urbino, C. del Valle Zeno.

PROCUNIAR, ROBERT WILLIAM, Dayton, Ohio. (Age 28.) With City of Dayton, Ohio. Refers to G. F. Baker, E. O. Brown, J. F. Hale, M. Hay, W. W. Morehouse.

REES, HAROLD, Cheyenne, Wyo. (Age 26.) Refers to R. D. Goodrich, H. T. Person.

SALISBURY, EUGENE FRANKLIN, Minden, La. (Age 47.) Chf. Engr., Louisiana & Arkansas R. R. Refers to C. N. Kast, C. S. Kirkpatrick, H. D. Knecht, A. A. Miller, R. Owen, R. L. Tatum, W. H. Vance, S. L. Wonson.

SCHOLTZ, WALTER, Los Angeles, Cal. (Age 22.) Asst. Shop Foreman, New England Pacific Screw Metal Products Co. Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas.

SHEARER, HERBERT LEWIS, Lexington, Ky. (Age 42.) With Central Rock Co. Refers to W. J. Carrel, J. F. Grimes, J. W. Guyn, R. B. Hayes, D. V. Terrell, J. S. Watkins.

SUTER, MAX, Urbana, Ill. (Age 44.) Graduate student, Univ. of Illinois. Refers to O. H. Ammann, H. E. Babbitt, J. S. Crandell, J. J. Doland, M. L. Enger, H. F. Ferguson, W. D. Gerber, W. D. Hatfield, W. C. Huntington, G. W. Pickels.

TAYLOR, JAMES ELLIOTT, Seminole, Tex. (Age 28.) Res. Engr., Texas Highway Dept. Refers to O. V. Adams, G. R. Johnston, H. N. Roberts.

TRAVIS, WAYNE IVAN, Boise, Idaho. (Age 25.) Jun. Engr., U. S. Geological Survey. Refers to I. C. Crawford, R. G. Kasel, T. R. Newell, C. G. Paulsen.

VALENSTEIN, SAMUEL, New York City. (Age 42.) Chf. Engr. with M. Shapiro & Son, Engrs. and Contrs. Refers to W. J. Ash, H. G. Balcom, A. J. Bernstein, H. H. Cashdan, N. A. Richards, O. Singstad, D. B. Steinman.

VAN HASSELT JACOB ADRIAAN KAREL, Yellow Springs, Ohio. (Age 51.) Prof. of Civ. Eng., and Acting Head of Dept. of Eng., Antioch Coll. Refers to A. W. Buel, N. C. Grover, S. Johannesson, W. J. Scott, H. Stabler, R. H. Suttie.

VIZCARRONDO, JULIO EUGENIO, Santurce, Puerto Rico. (Age 25.) Contr. Refers to J. Benitez-Gautier, A. S. Lucchetti-Otero, F. Pons, S. Quinones, R. Ramirez, A. S. Romero, E. Totti y Torres, C. del Valle Zeno.

WRIGHT, ROBERT ERNEST, Sierre Madre, Cal. (Age 40.) Engr., Standard Oil Co. of California. Refers to G. Boschke, C. E. Cate, G. R. G. Conway, H. Nunn, E. K. Smoot.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

CRAGGS, CLYDE, Assoc. M., Chicago, Ill. (Elected Jan. 17, 1927.) (Age 36.) Chf. Engr., Worden-Allen Co. and Permanent Constr. Co. Refers to H. H. Brown, G. Bryan, Jr., J. R. Fordyce, E. W. Kelly, C. Older, V. C. Ward, P. A. Zahorik.

HOUGH, FLOYD WOODWORTH, Assoc. M., Banning, Cal. (Elected Junior June 1, 1920; Assoc. M. Oct. 12, 1925.) (Age 35.) Engr., Colorado River Project, Metropolitan Water Dist. of Southern California. Refers to G. E. Baker, E. A. Bayley, J. B. Bond, W. Bowie, R. B. Diemer, C. L. Garner, E. O. Heaton.

KURTZ, ROBERTO, Assoc. M., Canpana, Buenos Aires, Argentine Republic. (Elected Oct. 12, 1925.) (Age 55.) Member of firm, Demartini & Kurtz, Engrs. and Constrs. Refers to R. Dominguez, D. P. Krynine, A. B. Lea, E. Lenhardtson, J. A. Valle.

MEAD, WILLIAM HENRY, Assoc. M., Luling, Tex. (Elected Oct. 9, 1917.) (Age 46.) Chf. Engr. and Gen. Supt., Salt Flat Water Co. and Darst Salt Water Co. Refers to H. Culpeper, R. J. Cummins, J. M. Howe, J. C. McVea, E. N. Noyes, F. H. Shaw.

MONTZ, JOHN McGILL, Assoc. M., Columbus, Ohio. (Elected Feb. 25, 1924.) (Age 48.) Associate Prof. Civ. Eng. Dept., Ohio State Univ. Refers to J. B. Jenkins, H. A. Lane, C. T. Morris, J. C. Prior, J. S. Sewell, C. E. Sherman, R. C. Sloane, E. Stimson.

FROM THE GRADE OF JUNIOR

GROOMS, ROY SEWELL, Jun., Decatur, Ala. (Elected Nov. 14, 1927.) (Age 32.) Designer, Alabama State Highway Dept., First Div. Refers to C. A. Baughman, H. D. Burnum, J. A. C. Callan, H. R. Creal, W. H. Fisher, P. D. Gillham, G. H. Sager, Jr., N. Williams.

HENNY, ARNOLD LORENTZ, Jun., Denver, Colo. (Elected Nov. 10, 1930.) (Age 30.) With U. S. Bureau of Reclamation. Refers to I. E. Houk, H. R. McBirney, B. S. Morrow, J. L. Savage, B. W. Steele, J. C. Stevens.

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VERANTH, JOSEPH, Jun., Ely, Minn. (Elected April 7, 1930.) (Age 32.) City Engr. Refers to E. F. Brownell, O. E. Brownell, G. H. Butler, Sr., R. W. Crum, J. P. Gebhard, L. M. Martin, S. W. Tarr.

WICKHAM, JOSEPH JOHN, Jun., Scranton, Pa. (Elected June 6, 1927.) (Age 32.) Textbook Writer in School of Civ.,

Structural and Concrete Eng., International Correspondence Schools; Instructor, Pennsylvania State Coll. Extension School. Refers to S. Baker, F. C. Foote, J. A. Fulkman, W. Jupenlaz, R. T. Lassiter, A. W. Skilling, E. S. Taub.

WOLFF, WILLIAM ROBERT, Jun., New York City. (Elected Nov. 11, 1929.) (Age 27.) Water Engr., Public Service Comm., State of New York. Refers to E. H. Aldrich, F. P. Gilbert, A. H. Pratt, E. H. Rockwell, W. M. Van Wagner, E. L. Walker.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

Board of Direction Meetings

January 15-16, 1934

A Quarterly Meeting will be held at New York, N. Y.

Annual Meeting

New York, N. Y.

January, 17, 18, 19, and 20, 1934

January 17, 1934:

Morning.—Annual Meeting. Conferring of Honorary Membership, and Presentation of Medals and Prizes.

Afternoon.—General Meeting.

Evening.—President's and Honorary Members' Reception and Dance.

January 18, 1934:

Morning.—Technical Division Sessions.

Afternoon.—Technical Division Sessions.

Evening.—Entertainment and Smoker.

January 19, 1934:

All-Day Excursion to Rockefeller Center.

January 20, 1934:

Inspection Trips.

The Reading Room of the Society is open from 9:00 A.M. to 5:00 P.M. every day, except Saturdays when it is closed at 12:00 M. It is closed all day on Sundays and holidays.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 267 current periodicals, the latest technical books, and the room is well supplied with writing tables.